PROCEEDINGS
THE INSTITUTION OF CIVIL ENGINEERS

PART I
NOVEMBER 1956

ORDINARY MEETING

15 May, 1956

WILLIAM KELLY WALLACE, C.B.E., President, in the Chair

The Council reported that they had recently transferred to the class of

Members

ENGHOLM, FRANK GOLDIE.
JONES, JOHN KELVIN, B.Sc. (*Wales*).
LAYTON, GEORGE THOMAS.
MATTHEWS, DENIS DEARMAN, M.A. (*Oxon*), M.Sc.(Eng.) (*London*).
MYLES, GORDON THOMAS, B.A., B.A.I. (*Dublin*).
OWEN, STANLEY FELIPE.
PICK, ZIGMUND.
SALMON, ARTHUR BASIL.
SMITH, WILLIAM MCCULLOCH.
TAYLOR, ALFRED LESLIE, B.Sc.(Eng.) (*London*).
WATTS, HENRY ROBERT, B.E. (*New Zealand*).
WILSON, JAMES KELMAN CUNNINGHAM.

and had admitted as

Graduates

ANDREWS, ROBERT ALEXANDER, B.Sc. (Eng.) (*Rand.*).
BEVAN, JOHN GOWER, B.Sc.(Eng.) (*London*), Stud.I.C.E.
BOWIE, JAMES DOUGLAS, B.Sc. (*Glasgow*).
BURR, GRAHAM ERSKINE, B.Sc. (*Aberdeen*).
CHEAH THEAN SUN, Stud.I.C.E.
CROCKER, MARTIN CECIL, Stud.I.C.E.
DICK, JOHN JACK, Stud.I.C.E.
EVANS, ROLAND THOMAS, B.Sc.(Eng.) (*London*), Stud.I.C.E.
EVERETT, RICHARD WILLIAM, B.E. (*New Zealand*).
GADD, MICHAEL LAURENCE, Stud.I.C.E.
GINANAMUTTOO, YOGANANDAN MITCHELL, B.Sc. (*Ceylon*).
GRIFFITHS, DONALD MICHAEL, B.Sc. (*Wales*), Stud.I.C.E.
HANNAN, RALPH ADRIAN, B.Sc.(Eng.) (*London*), Stud.I.C.E.
HOLT, JAMES ALEXIS NEVILLE MACPHAIL, B.Sc. (*St Andrews*).
HUTTON, HERBERT ROBINSON, B.Sc.(Eng.) (*London*), Stud.I.C.E.
KOH KOK EE.
KWOK, KENNETH WAI KAI, B.Sc. (*Hong Kong*).
LEACH, GEOFFREY STEWART, B.Eng. (*Liverpool*), Stud.I.C.E.
LEVI, HENRY LUDWIG, B.Sc. (*Cape Town*).
LEVINGSTON, IAN ALEXANDER CEDRIC, B.A., B.A.I. (*Dublin*).
MAY, MICHAEL JOHN, B.Sc.(Eng.) (*London*), Stud.I.C.E.
PLAUT, RUDOLF THEODOR FELIX, B.Sc. (Eng.) (*London*), Stud.I.C.E.

RITCHIE, HENRY GEORGE, B.Sc. (*St Andrews*).
SELVESTER, GERALD AUSTIN, B.Sc. (*Birmingham*).
TAYLOR, NEIL, B.Eng. (*Liverpool*), Stud.I.C.E.
WEBBER, MICHAEL HANSFORD, B.Sc.(Eng (*London*), Stud.I.C.E.
WILD, JOHN PATRICK, B.Sc. (*Leeds* Stud.I.C.E.

and had admitted as

Students

ACFORD, IAN HAYCROFT.
AKINTOYE, RAPHAEL OLANIYI.
ALDER, RICHARD.
BENFORD, EDWARD JOHN.
BRIGGS, ALAN.
BUDDS, RICHARD WILLIAM.
BURTON, ANTHONY RICHARD.
CLEARY, MICHAEL.
COLLEY, JOHN REDFERN.
COLLINS, LOUIS WILLIAM.
DALGLEISH, QUINTON WILLIAM.
DUMBRILL, GRAHAM JAMES LEONARD.
DYER, JAMES.
ELAND, JOHN BORDER.
EUYEN, EDUARD JOHAN ANTON.
FEY, JOHN ANTHONY.
FISHER, DAVID IAN.
FRENCH, TERENCE EDWARD.
GIRLING, MICHAEL THOMAS.
GLAISTER, BARRY MICHAEL MURRAY.
HALL, VICTOR FRANK.
HARDY, ROBERT.
HARRIS, CLIVE MALCOLM.
HEUNIS, CASPER.
HUNTER, JOHN RUBLEY.
JOHNSTON, KEITH HENRY.
KHAN, NASRULLAH.
KYARI, MOHAMMED MAYINTA.
LITTLE, JAMES STEPHEN.
MCKENNA, HUGH.
MALTMAN, JAMES.
MAYNARD, EDWARD JAMES.
MOORE, HUGH.
NASSIM, VICTOR HASKELL.
ROBINSON, JOHN PHILIP.
SAVAGE, JOHN ANTHONY WILLIAM.
SEARCHFIELD, GARY THOMAS.
SHAW, COLIN.
SKELDON, ARNOLD GEOFFREY.
SMITH, IAN WARBURTON.
SOYANNWO, SOFOLUWE OLUSOLA.
SWIFT, DEREK.
TURNER, BARRY.
WALLS, TERRY.
WHATLEY, PETER WILLIAM.
WHITBREAD, FREDERICK JOHN.
WHITE, BRUCE RODNEY.

The President said that when the awards to Authors of Papers presented Session 1954/55 had been presented to the Authors at the last Ordinary Meetin Mr W. R. Schriever had been unable to be present owing to his being resident Canada. Mr R. F. Legget, Member of Council for Canada, was present tonight an had undertaken to accept, on Mr Schriever's behalf, the Baker medal which ha been awarded to him for his Paper on "Strain measurements on the temporary roa deck for the Toronto Subway".

The President then presented the Baker medal to Mr Legget.

Mr J. T. Evans moved, and **Mr A. L. Little** seconded, the following resolu tion:—

"That Messrs H. M. Bostandji, H. R. Boyce, D. A. Brown, R. Carey, E. V Cuthbert, R. W. A. Fane, A. B. Henderson, E. C. Lightbody, and H. Ridehalgh be appointed to act as scrutineers, in accordance with the By-laws, of the Ballo for the election of the Council for the year 1956–1957."

The resolution was put to the meeting, and carried.

JAMES FORREST LECTURE, 1956

The President said that the James Forrest Lectures had been established in 1891 at the wish of James Forrest, who had been Secretary of the Institution from 1859 to 1896, and Honorary Secretary until his death in 1917. The original endowment had been the balance of a sum of money subscribed by Members of the Institution for engraving the portrait of Mr Forrest; to that endowment Mr Forrest had added a similar sum by bequest to the Institution, in order to establish a series of Lectures. He had also bequeathed to the Institution a number of pieces of presentation silver, of which he had been the recipient during his Secretaryship. This evening marked the sixty-second Lecture of the series.

The President then announced that the sixty-second Lecture would be delivered by G. E. R. Deacon, C.B.E., D.Sc., F.R.A.S., F.R.G.S., F.R.S.

Dr Deacon was born in 1906 and had been educated at King's College, London. He had served on the scientific staff of the Discovery Committee in England and in the Royal Research ships *William Scoresby* and *Discovery II*, 1927–1939. He had been awarded the Polar Medal in 1942. He was Director of the National Institute of Oceanography.

The President then called on Dr Deacon to give his Lecture, the subject being "Marine physics".

MARINE PHYSICS

by

George Edward Raven Deacon, C.B.E., D.Sc., F.R.S.

Introduction

I should like to thank you for the honour you have done me by inviting me to give the James Forrest Lecture, particularly since I believe my subject affords a good example of the interdependence of abstract science and engineering.

The oceans cover the greater part of the globe; they afford a wealth of food and chemicals; we spend millions every year improving our use of them and our defences against them; we prospect for the minerals they have deposited; they are an important part of the machine which controls climate and rainfall, and it is no exaggeration to say that our dependence on them increases with the growing world population and standard of living.

There has always been a challenge to understand, predict, utilize, and control them. It would be interesting and useful to follow the development of marine science from the days of the early navigators and scientists, but to allow time for a discussion of modern trends we might begin with the situation as it was seen by Admiral W. H. Smyth just over 100 years ago. Speaking with the authority of a distinguished career and service in the high offices of a number of our learned societies and the United Services Institution, he wrote "the various branches of available science have been so steadily advancing among seamen of all nations that besides higher practice in mechanical navigation they possess a more accurate information respecting the phenomena of winds and oceanic currents than heretofore. Already the elements are nearly reduced to subjection by the union of science and practical seamanship so that sea passages are wonderfully shortened within memory".

Such promise has scarcely been maintained. There have been tremendous

improvements in charts, coastal descriptions, and navigational manuals and Tab
There has been a great advance in information of winds and currents, but lil
increase in understanding of the relations between them, and anyone seeking
further some aspect of navigation, fisheries, coastal engineering, or marine metec
logy by applying a knowledge of the forces which govern any kind of water movem
is sure to find the available information inadequate for his purpose. The rewa
are not so spectacular as they were 100 years ago, but they are probably more s
stantial. The problems are too complex and intractable to be solved by sim
charting of winds and currents, but they are proving amenable to modern phys
methods. Progress is slow because they do not receive sufficient attention; too
expert physicists are aware of the problems, and too few authorities realize t
science can help. Sailors, for example, feel that experience at sea will do more
increase our knowledge of ocean currents than anything a scientist can do, yet
amount of such experience will allow precise account to be taken of wind str
friction, and the earth's rotation which are the essential factors. Without sou
theory we cannot do more than make a statistical analysis which gives little or
information about significant variations from day to day and place to place.

As an example of the neglect of marine physics, it may be fair to quote the co
prehensive work "Modern Physics for the Engineer".[1] It devotes 36 pages
astrophysics and 24 to the earth below the sea, but not a word to the seas themselv
The idea behind the book is that modern engineering rests on a base of experimen
and theoretical work in the sciences as well as on the work of centuries of enginee
It holds that the physics of today may be the engineering of tomorrow. Since t
contents were planned by a committee of eminent engineers and scientists, t
omission of marine physics might imply that they considered it insignificant a
unpromising. On the other hand, they may have thought it too large a subje
developing too quickly to allow a concise account; but they made no apology.

To show respect for the book and the idea behind it, an attempt to "build u
marine physics may well start with the same quotation from Ernst Mach [2] "Th
part of Physics which is the oldest and simplest, and is therefore to be considered
the foundation for the understanding of many other parts of Physics, concerns its
with the investigation of the motion and equilibrium of bodies". Such a st
would be appropriate; the foundations of our study of the oceans are very shak
and it is time to consider whether more sound physics at the base will not make t
rest of the work more rewarding.

Sources of energy

Most of the energy in the sea is derived from solar radiation. There is no eviden
that the earth as a whole grows warmer or colder, and it must be assumed that t
total amount coming in is balanced by the outgoings. About half of the radiati
reaching the earth is reflected from the clouds and the sea and land surfaces. Som
thing like half of that which gets absorbed is returned as long-wave radiation; t
remainder is communicated to the lower atmosphere, chiefly by the processes
evaporation and condensation. Finally it is radiated away from the atmosphere.

But although the ingoing and outgoing radiation is balanced over the whole eart
taking one area with another and one season with another, there are significa
differences in the regional and seasonal exchanges. The balance is achieved on
with the help of large transports of heat in both atmosphere and ocean, mainly fro

[1] The references are given on p. 674.

ow to high latitudes. The processes, particularly the interchanges between the oceans and atmosphere, are complex. The atmosphere absorbs little direct radiation from the sun, so that it is heated mainly from below and cooled from above. In the oceans the heating and cooling both occur at the same level, so that the thermodynamic efficiency, for turning heat into motion, is much less in the oceans than in the atmosphere. It has been estimated that the total kinetic energy in the sea is not more than 2% of the kinetic energy in the atmosphere, and most of this 2% appears to be derived from the atmosphere through the drag and pressure of wind on the water. The ocean currents driven by the winds do much to determine where most heat is fed to the atmosphere, and so help to set the pattern of cloudiness which influences the amount of incoming radiation. The winds and water movements influence the density layering of the oceans, altering its thermodynamic potential as well as its response to the winds. There is no simple reasoning; so far as we know the winds make the Gulf Stream, and the Gulf Stream makes the winds. More detailed understanding of the complex interchange is as necessary for meteorologists as for oceanographers, and it is being pursued by increasingly active collaboration between them.

There are other smaller sources of energy. The heat conducted through the sea bottom is perhaps a thousandth part of that absorbed at the surface, and although it is supplied at the most effective part of the heat engine it is generally supposed to have no appreciable effect. The heat produced by oxidation of organic matter and by other chemical processes in the sea is again believed to have no measurable effect. Another small but very effective source is the work done on the oceans by the earth, moon, and sun as they move relative to one another.

The tides

Every now and then there is a complaint in one of the professional journals that it is easy to see how the moon can diminish the force of gravity on the part of the earth directly underneath her, and incredible that there should be a similar effect on the side farthest away. The complainant is usually satisfied about the tidal "hump" on the side of the earth facing the moon, but very worried about the one on the other side. The Director of the Liverpool Observatory and Tidal Institute has helped on such occasions by emphasizing that two main principles must be borne in mind:

(1) only the variations in attractive forces of the tide-generating bodies need to be considered, for if all parts of the earth were to experience equal forces there would be no movement relative to the earth; and

(2) components of the attractive forces vertical to the earth's surface cannot raise tides because they are so small in relation to the earth's own gravitational force—it is the tangential or horizontal components that raise the tides.

The rest is simple; the rigid earth can be considered as subject to the force at its centre, whereas the water at points on the earth's surface is subjected to the attractive forces operating there. If these were equal to the force at the centre there would be no differential force to cause relative movement, but the forces at the earth's surface nearer the moon are greater than at the centre, and this is greater than the forces at the earth's surface away from the moon. The horizontal components of differential force are therefore directed towards the line of centres of earth and moon, towards the side of the earth farthest away from the moon, as well as to that nearest the moon. It is not so obvious that the centrifugal forces due to the movement

of the earth and moon round a common centre of gravity must be the same at points on the earth's surface, but it is understandable when one thinks of a n rotating earth moving round an axis fixed in space, It follows that when attractive forces are stronger than the average they overbalance the centrifu force, and where they are weaker than the average they are overbalanced by it

This simple theory, based on an earth completely covered with water, is suffici to show how the tide-raising forces at any point on the earth's surface depend on variations in the relative positions of the earth, moon, and sun, and why there semi-diurnal as well as diurnal tides. It is not sufficient to explain the response all the seas and oceans of different size, shape, and depth. The application of ti theory to these realistic problems becomes exceedingly complicated. Fortunat it is not necessary to be able to calculate from first principles what the tide sho be at any point. The tide-raising forces themselves are known with such accura that the expert knows what periods and phase relations to look for in a tidal rec If, therefore, a sufficient length of record to show the effect of the varying force available, careful analysis and reconstruction in relation to future astronomical d will allow accurate predictions.

The fact that such reliable predictions can be made does not remove the need further research. Records cannot be made in every place for which predictions required, and to make the best use of a short run of observations or of measureme some distance away requires a detailed understanding of the nature of the ti oscillations. The measurement and prediction of tidal streams is not as easy as th of tidal heights, and there are particularly difficult problems in estuaries and shall water. One of the most outstanding needs is a more detailed understanding of disturbance of the tides by winds and variations in atmospheric pressure. T could be used to assist navigation, and to assess the probability of coastal flo and improve warning against them.

It is safe to say that few subjects are so often misunderstood as the tides, yet f have been so carefully presented by scientists from Newton onwards. Unsatisfact "explanations" still appear in books intended for elementary students; one of favourite errors is that the tides are all generated in the Southern Ocean, an id which was shown to be untenable 100 years ago.

Waves

Our knowledge of the tides is much more satisfactory than that of other wa movements in the sea. This lead has been established because the tide-raisi forces obey well-known laws; they can be specified much more precisely than wi stresses, and calculated years ahead. Accurate continuous records of the tides available too, since they can be obtained much more easily than records of wa and currents. These advantages have made the study of the tides more approacha and more attractive to academic workers.

In contrast, the physical processes by which wind blowing over water genera waves are too complex and uncertain to allow a precise theoretical approa Theoretical work has shown that the sea surface must become unstable, but it d not explain the formation of the characteristic wave patterns or help to relate th dominant features to the strength and duration of the wind.

Wave predictions have to be based on semi-empirical methods and they are mu more empirical and less rational than the methods for tide predictions. The for change unpredictably in consequence of many interdependent factors and, even

they could be measured accurately, the difficulties which have to be faced in predicting their effect on actual ocean surfaces are as serious for waves as for tides. We need to know the relative extent to which energy may be communicated to the waves by the direct pressure of wind on the windward slopes and by the frictional drag of the wind over the wave profile, and how these processes vary with the development of the waves. We also need to know how energy is transferred from short to long waves as the wave pattern develops from a short choppy sea to one dominated by great storm waves. Another difficulty is our poor understanding of the processes which damp down waves and swell as they travel through cross winds, tidal streams, and areas of irregular bottom topography. When we can assess the effect of these factors we shall have a universal method of prediction, but until then a method of wave prediction developed for one place and range of weather conditions may not be very satisfactory when used in other places. Since the task of making representative series of observations at every place and in all weather conditions for which predictions may be required is out of the question, any improvement in our understanding of the basic physical processes will prove very rewarding.

Detailed study of the variations in air pressure very close to the surface of the waves is an example of the type of research that is needed. To obtain such measurements we have made a free-floating buoy which follows the profile of all but the smallest waves. It is shaped something like the top of a mushroom 5½ ft in diameter and 1½ ft deep. It weighs about 11 cwt. It contains a doubly integrated accelerometer to record wave height, and gyroscopes to measure wave slopes in the main direction of the wind and at right-angles to it, the source of direction being maintained with a small sea anchor. The buoy contains a specially designed microbarograph communicating with the air through groups of small holes in the flat top of the buoy about 3 in. above the surface of the water. At low wind speeds the records from the microbarograph and wave-height recorder follow each other very closely and the microbarograph registers no more than changes in altitude. At high wind speeds the variations in air pressure are much more irregular than the recorded wave profile, and now we are faced with the rather difficult task of analysing a representative series to find possible systematic relations in amplitude and phase.

Most of our methods of wave recording were described in a lecture given by Barber.[3] To obtain observations from the open ocean we have had to supplement them by a remarkable instrument which solves the very difficult problem of recording wave profiles from a moving ship. This shipborne wave recorder combines continuous measurements of pressure through a small hole in the hull of the ship with simultaneous measurements of the up and downward movements of the hole, obtained by doubly integrating the output of a vertical accelerometer. The design of the instrument and integrating circuits allows for the rotary accelerations in the waves, and recordings from two instruments, one on each side of the ship, are averaged to avoid errors due to the reflexion of waves from the ship's side. Such recorders have been used in the R.R.S. *Discovery II* for 3 years and in the ocean weather ship *Weather Explorer* for 2 years. They have since been fitted in three research vessels in the United States and in the *Norsel* chartered by the French Antarctic Expedition. There are now a large number of records from deep water. The highest waves measured so far are a little over 50 ft. A detailed account of the instrument has recently been given to the Institution of Naval Architects by M. J. Tucker.

As more accurate and continuous records become available the methods of wave prediction improve, but there are still a number of different approaches to the

problem. Each is founded on empirical rules and reasonable physical assumptio In this country Darbyshire [4] (1952) developed a method for the north coast Cornwall, and later [5] (1955) modified some of his conclusions to account for higher and steeper waves measured in the open ocean. He suggests that the diff ence can be attributed to the greater effect of turbulence near the coast where depth of water is of the same order as the wavelength and there are active ti streams. The most recent survey of existing methods is by Gelci, Cazale, a Vassal [6] (1956) for waves and swell at Casablanca; they argue the relative merits the physical assumptions made by Darbyshire [4] (1952), Pierson, Neumann, a James [7] (1953), and show how to make good predictions for Casablanca.

One of the most interesting advances is the study of the statistical scatter of wa heights. Barber and Ursell [8] (1948) showed that the waves behave roughly though they originated in a number of point disturbances distributed over t generating area. This implies a continuous spectrum of waves and a wave patt that at any time is the sum of a large number of component wave trains. Th showed that the component waves travelled independently after they left the sto area, and explained why the wave period declined slowly as the components of shor period followed the longer fore-runners to the coast. The distance of the sto could be inferred from this trend of wave periods, and a number of examples we substantiated with the help of the relevant meteorological charts. In his lectur Barber showed how the interference of a number of wave components explained t great variation in wave height which is observed when waves are watched or record over an interval of 10 to 20 min. By analogy with acoustics the groups of high a low waves are "beats" developed between incoming trains of different frequen When an active wind is driving the waves and there is a broad spectrum, high a low waves follow each other and the time intervals are irregular. In contrast, sw which has travelled some distance from the storm area has a narrow spectrum a there are more gradual rhythmic changes. He worked out the probability d tribution on the assumption that the departures of the water level from the mean a distributed according to normal error laws. This is much the same as assuming th there are only random phase relations between the component wave trains. It l to the conclusion that 10% of the waves will be higher than 1·7 times the avera and 10% lower than 0·34 times the average. More detailed studies, including t extension of the work to the two-dimensional problems which arise when there interference between wave systems from different directions, have now been ma by M. S. Longuet-Higgins and others. The results show how simply observab properties, such as the intervals between a series of successive maxima or the orient tion of the surface contours, can be used to obtain useful information about t wave spectrum and the spread of wave directions.

The statistical theories assume that the contributions from different parts of t generating area are superposable. This assumption can be shown to be valid for lo waves, and the theories should not apply to waves near their maximum heig The good agreement between the theory and measurements made at sea seem however, to give a clear indication that it gives a satisfactory representation of very wide range of sea conditions. One disconcerting consequence revealed by t statistical studies is that two ships lying a mile or so apart are likely to obtain wa records which look remarkably different. If the wave spectrum is narrow the mea wave heights evaluated from 10-min records may easily differ by as much as 25% The probable error becomes smaller as the spectrum gets wider. It means that 10-min record is not a very accurate guide to what will happen during the ne

10 min or to what is happening during the same 10 min a mile or so away. Longer records will give better agreement, but they cannot be too long or changes in the mean state of the sea will affect them.

The new information about the energy spectrum of waves and the methods used in the statistical studies of waves are proving useful in the study of ship motion. There may still be naval architects who are compelled to study the effect without understanding the cause, but the wave studies will show how the probabilities of occurrence of maximum amplitudes, velocities, and accelerations can be calculated from the energy spectrum, and inferred with reasonable accuracy from meteorological records. They will increase the value of model experiments by showing how to apply the conclusions to realistic wave patterns.

The physical problems at the edge of the sea are more difficult than those in deep water, but useful progress has been made with theoretical studies of the water transport associated with waves in shallow water. Engineers who deal with problems such as the removal of beach material and the deposition of silt in estuaries have built up a great wealth of experience and pointed out many interesting problems. They are very complex because there are so many interdependent factors but modern physical methods will allow useful progress to be made. Since the amount of sediment transported depends very critically on the speed of the water movement it is particularly important to know how the water movements depend on the waves and the wind.

Rip currents are striking instances of beach currents made by waves. Waves tend to carry water towards the beach and to establish a slight head of water, sufficient to produce compensating currents out to sea. The backward flow tends to concentrate where there are depressions leading out from the beach, forming outgoing rips which can be dangerous to bathers. They have been demonstrated in models so that there is no doubt that high waves as well as tides can produce dangerous currents. They are usually narrow, and a bather caught in one can probably escape by swimming along the beach. Some of the danger might be averted if we knew more about them.

The pressure variations below ordinary progressive waves decrease rapidly with depth, but below standing waves there are pressure variations of half the wave period which do not decrease with depth. There is never a simple standing-wave pattern, but "standing seas" are often seen between two opposing wind and wave systems, and "pyramidal waves" are produced by waves blowing into the centre of a cyclone. Longuet-Higgins [9] (1950) has made a careful theoretical examination of the problem and has shown that there is sufficient formation of standing waves in such circumstances to set up compression waves in the sea and in the sea-bed, giving rise to regular 3- to 10-sec ground oscillations which can be detected at distant seismological stations. Similar pressure variations can also be produced where the waves reflected back from a rocky coast interfere with the oncoming waves. Darbyshire [10] (1950) has identified recordings of microseisms at Kew with wave activity in the middle of the Atlantic Ocean as well as with waves reaching the coast of Cornwall. Cooper and Longuet-Higgins [11] (1951) have demonstrated the second-order pressure variations below standing waves produced by reflexion from a beach in a model tank. This study of microseismic oscillations is useful because it may be possible to obtain information about oncoming swell from the ground waves which arrive long before them. It is also possible that measurements of the bottom pressure variation may afford a useful measure of the reflexion of wave energy from beaches. The problem of microseisms is one which has interested seismologists since Milne [12] dealt with it in 1881, and this new solution affords a good example of the effectiveness of modern

physical methods. I ought to mention that there are other ground movements addition to the 3- to 10-sec oscillations which have been considered the main probl so far, and to whisper that there are still a few seismologists who do not yet beli the elegant and promising solution afforded by the new physical approach.

LONG WAVES

There are a number of types of sea-surface oscillations intermediate between t ordinary waves and the tides. On any ocean beach it can be noticed that ev 2½ min or so the uprush from the breaking waves comes higher up the beach than has during the past 2½ min. It has been found that there are 2- to 5-min periods the envelope of the incoming swell, an imaginary smooth line drawn over the incomi groups of high and low waves. These long waves were aptly called surf beats W. H. Munk, who observed them on the coast of California. A brief account of th is given by Barber [8] and a more detailed study by Tucker [13] (1950). Mr Tuck measured long sea waves of 2- to 3-min period and a few inches in amplitude. Th may be partly responsible for energizing natural oscillations in harbours, known range action.

The natural oscillations of gulfs, bays, and larger areas of water such as a co tinental shelf are generally of longer period. They are often noticeable on tid gauge records, giving a marked saw-tooth appearance to what is at other times smooth harmonic curve. Such oscillations have been studied extensively in lak and the name seiches which has been applied to them is also used for the sam phenomenon in sea basins. The recorded oscillations are usually complex, wi movements from side to side as well as end to end, and with oscillations of sm sections superimposed on those of larger areas. They are set in motion by any di turbance of the water, and atmospheric disturbances are the most frequent cause.

We know little about the propagation of long waves in the deep ocean. Observ tions at Bermuda (Redfield and Miller,[14] 1955) indicate that the elevation of the s surface during the passage of a hurricane is due primarily to the reduced baromet pressure, and any accumulation of water due to the wind should soon be dissipate There is, however, some evidence that a travelling depression can cause long-wa activity on a distant coast, without approaching it. Imbert [15] (1954), using pressure recorder on the coast of Adélie Land, south of Australia, found long-peri oscillations although the ordinary waves were prevented from reaching the lan by at least 100 miles of pack ice.

In shallow water there are more striking changes in sea level. One type occu when the disturbance travels at the same speed as a free progressive wave, so th the wave is continually energized. In a depth of 50 fathoms this requires the stor to move at 60 knots and in 25 fathoms at 42 knots. Frictional forces prevent undu large responses, but surges generated with this kind of resonance in the Englis Channel have occasionally caused a good deal of minor damage and confusion amon small craft beached or moored along the coast, and they are well known in oth shallow seas. There is also evidence that long waves travel behind a moving depre sion in the same way that waves travel behind a ship, their period and length bein determined by the speed of the depression. There are also complex effects due t water driven by the wind being checked and piled up against a coast. The mos destructive floods have occurred where a hurricane or typhoon has moved rapidl across an expanse of shallow water.

W. H. Munk has emphasized the possible effect of another type of oscillation foreseen by early theoretical workers and called by Lamb [16] (1932) edge waves. The energy associated with the ordinary pattern of waves running on to a sloping beach is shown to produce another type of wave which moves along the beach with its crests at right-angles to the shore. Its wavelength and period depend on the slope of the beach and the amplitude becomes negligible at a distance of one wavelength from the beach. Those produced by ordinary waves are believed to play a part in the formation of beach cusps, but those due to long waves, depending on the gentler mean slope of the bottom extending up to 50 miles or more from the coast, have periods which may be as much as several hours. Munk, working on the coast of California, has measured 10- to 30-min waves. The periods and the decrease in amplitude away from the coast agree with the edge-wave theory. Any form of long-wave energy approaching the coast is likely to cause edge waves resonating with the local topography—the deep sea disturbance is in effect trapped into much slower oscillations. Long-wave recorders on the coast of California show as much as 5 days' activity after a major seismic disturbance, and although some of the prolongation is likely to be due to multiple reflexions and scatter from the continental beaches and main topographical features, Munk suggests that the transformation of fast open sea waves into slow edge waves may also be important.

The study of long waves has a number of practical applications. Better understanding of the processes by which troublesome oscillations in harbours are energized would allow prediction of the worst occurrences and facilitate control. More understanding of the meteorological disturbance of tides would soon allow prediction which would be of very direct benefit to navigation. Research on storm surges will allow earlier and more reliable warnings to be given. Those now given by the Flood Warning Organization based on the work of Corkan, Doodson, Proudman, Rossiter, Farquharson, Suthons, and others are remarkably good, but a better understanding of the physical processes is needed to make the methods more generally applicable, and to give a better idea of what maximum heights are likely to occur in different places. The theoretical work is difficult. A recent example is the work of Crease [17] (1956), which shows that the effect of the earth's rotation on long waves approaching the north of Scotland from the Atlantic Ocean will produce waves travelling southwards behind the barrier formed by Scotland and England, and although the height of these waves decreases exponentially from the coast it may be considerably more than the Atlantic waves along the coast itself. This type of problem requires a real appreciation of the principles of fluid dynamics. The necessary measurements are not easy to make since the astronomical tides have to be subtracted from the general water-levels.

Ocean currents

Our present understanding of wind drift is of a much too general nature to allow ocean currents to be predicted from meteorological charts and forecasts. Thanks to the painstaking work of marine observers, especially since the middle of the nineteenth century when the systematic collection of data was organized on an international scale, we have well-documented charts to show monthly averages for all parts of the oceans regularly traversed by ships, but the monthly charts often emphasize the variability of the current, and good use could be made of principles which would allow reliable estimates of the speed and direction of a current under observed or forecast wind conditions. To gain such a detailed understanding is much more difficult than the current charting done so far.

One of the first things to do was to make a detailed study of an ocean current, a
this was done for the Gulf Stream by oceanographical laboratories in the Unit
States and Canada. Early measurements in the current, 100 years ago, show
that it was unsteady, shifting its position, so that the set varied greatly in streng
and direction, particularly at the edges of the current. Since then the search fo
simple representation and the averaging of many thousands of observations h
tended to create a popular impression that the current is steadier, broader, a
weaker than it really is. New observations north of Cape Hatteras, after the curre
has parted from the coast, show that there are generally speeds as much as 4 or 5 kn
in a narrow region, about 10 to 15 miles wide, near the landward side of the curre
Transient horizontal eddies with diameters ranging from several miles to a f
hundred miles have been observed, and the research vessel *Atlantis* found a te
porary 3- to 4-knot counter current where the average chart showed the strong
forward movement should be. The new detailed surveys are made possible by ra
methods of position finding, and they are helped by new methods of measuri
currents. The most remarkable method infers the speed and direction of the wa
transport from measurements of the electric fields induced by the movements of t
water through the earth's magnetic field, measured by an electrode cable tow
behind a ship. The other major ocean currents seem to possess similar compl
structures, and the more variable drifts such as the North Atlantic drift betwe
Newfoundland and Europe seem to depend primarily on the local winds. Su
currents are subject to large "synoptic" changes superimposed on the gene
"climatic" pattern; the North Atlantic drift is a prevailing current only to the sa
degree that the prevailing wind is westerly. There are many synoptic situatio
where it flows the other way.

To obtain a satisfactory understanding we must study the drag of the wind on t
sea surface under different conditions of wind strength, fetch, and air stability. O
of the methods is to study the shape of the vertical profile of wind above the surfa
another to relate the slope of a water surface to the speed of the wind blowing acr
it, and Bowden [18] (1956) has studied the effect of changes of wind on the flow of wat
through the Straits of Dover. The method of monitoring the flow of water
to record the varying voltage between the English and French coasts due to t
varying flow of water through the earth's magnetic field. Another necessary task
to study the frictional processes, particularly near the sea-bed, and this involves th
detailed investigation of the turbulent fluctuations.

The new information which is being gathered begins to allow the existing theori
to be examined critically and to provide a basis for new work. Oceanographers a
no longer satisfied with the assumption that the horizontal pressure gradients ass
ciated with the slopes of density surfaces are exactly balanced by the geostroph
forces which result from movement along the contours of the density surfaces. Co
sideration is being given to the errors introduced if the current varies with time an
if its speed and direction change sharply along its path. Account is also being take
of the piling-up of water along a coast so that the slopes of the isobaric surfac
cannot be inferred reliably from the slopes of the density surfaces, and of the poss
bility of mixing across the density surfaces, and the effect of friction.

If the wind drag were the only force acting on the water the current would be
the same direction as the wind and the frictional resistance would be exactly opposit
Since every part of the earth's surface not on the equator has some rotation about
vertical axis, once every 24 hours at the poles, and slower towards the equator, an
water movement experiences a centrifugal force, sufficiently strong in relation

wind drag and friction to cause some deflexion to the right in the Northern Hemisphere or to the left in the Southern Hemisphere. As this deflected drift is transmitted downwards by friction it is deflected more and more. Ekman [19] (1905) studied this problem to explain Nansen's observation that Arctic ice drifted about 20–40° to the right of the wind. He assumed a homogeneous sea with a uniform coefficient of viscosity, and found that in deep water far from land the surface water would move 45° to the right of the wind in the Northern Hemisphere. Below the surface it turns more and more to the right until, at a depth which would be about 40 to 50 fathoms for a wind of 20 knots in mid-latitudes, the direction is completely reversed and the speed about one twenty-third of its value at the surface. This current system is known as the "Ekman spiral" (see Fig. 1). Statistical study

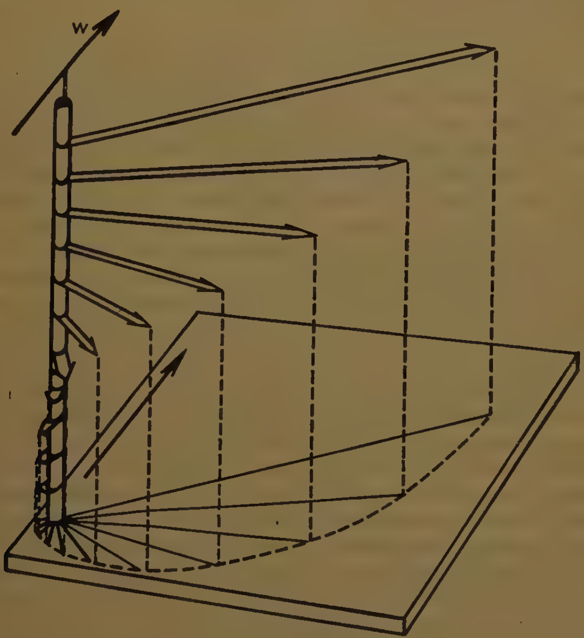

FIG. 1.—SCHEMATIC REPRESENTATION OF THE DECREASE IN VELOCITY AND CHANGE IN DIRECTION OF A WIND CURRENT WITH DEPTH

f surface current observation has afforded angles between the wind and water novement in good agreement with the 45° of the theory, and there is some evidence f the increasing deflexion with depth.

During the past few years Stommel [20] (1948), Munk [21] (1950), and others have nade further contributions to the theory of wind-driven circulation. One of the nost interesting advances is Stommel's explanation of why the currents on the estern sides of the oceans are stronger than those on the eastern sides. Account as to be taken of the increase in geostrophic forces with latitude. There is a continuous anticyclonic (clockwise) circulation of water round the North Atlantic Ocean ue to the prevailing wind system, and if this were balanced by frictional forces alone

the north and south currents on the west and east sides of the ocean would be simi The water in the northward current has an increasing clockwise spin relative to earth as it flows into higher latitudes where the earth has more and more anticlo wise spin. The effect of the earth's rotation is therefore to increase the clockw circulation in the west and to reduce it in the east where the water flows southwar This has to be balanced by anticlockwise frictional stresses which must be greate the west than in the east and such balance cannot be achieved unless the curren stronger and narrower in the west. Using such ideas expressed much more precis and reasonable assumptions about wind stress and friction, Munk has been able account for the principal features of the average circulation of the North Atla Ocean. The detailed structure of the currents is not yet explained and there is mu to do before we can relate the growth and changes in a current to the onset a variations of wind.

Deep-water circulation

The mechanism of the surface currents must be related to a large extent to tra port of water by deep currents. The existence of such movements has been infer since the early nineteenth century, when it was realized that the cold water found the bottom of the tropical Atlantic Ocean must have flowed there from the po regions. Ever since then it has been argued whether the water movements in t ocean are due to wind or convection. Those who favoured convection were on t whole ready to admit that the surface currents might be caused by the wind, b they could not see how friction of the wind at the surface could cause apprecia movements at great depths. This would be a difficulty if the oceans were of infin extent, but, since they are limited, water can be piled up against a coast and th can cause horizontal pressure gradients at great depths as well as near the surfa Horizontal pressure gradients may also be produced by the processes which increa or decrease the angular momentum of water as it flows from one latitude to anoth There is some evidence that the deep-water circulation in the Atlantic Ocean is mu more active than that of the Pacific Ocean, and although there are some clima differences between the two oceans the most obvious difference is that the Atlan Ocean is very narrow compared with the Pacific Ocean. The wind probably has important effect on the density distribution. If there were no wind the heating a cooling at the surface would produce some circulation, but it would probably limited to a shallow surface layer. In the actual oceans we find warm water pen trating to considerable depths, especially in the subtropical regions, and the actu density distribution can produce appreciable thermodynamic movements. B although the effect of climatic differences and horizontal density gradients may n be so important as wind stress, it must be considerable, and the density layering the ocean must have significant influence on its reactions to wind stress. There also evidence of water movements associated with oscillations (internal waves) layers where there are sharp vertical density gradients.

By making temperature observations and collecting water samples from repr sentative depths from the surface to the bottom at a sufficient number of poin along an ocean traverse, we can plot vertical sections showing the distribution temperature and salinity and obtain unmistakable evidence of movements of wat from one end of an ocean to another. Such sections from north to south in t Atlantic Ocean show that highly saline water sinks from the surface in the bounda region between Arctic and Atlantic currents near Greenland and Labrador. It joined by water of still higher salinity which flows as an undercurrent from t

Mediterranean Sea and spreads southwards across the equator at depths of 1,500 to 3,500 m. As it approaches the Antarctic it climbs steeply to within about 200 m of the surface where it appears as a rather warm saline layer below a colder less saline surface layer, and above a colder but only slightly less saline bottom layer. The cold waters spread north as well as east. In the Atlantic Ocean the Antarctic surface water forms the nucleus of a poorly saline layer, intermediate between the warmer and more saline surface waters and the North Atlantic deep current, and it can be traced as far as 25° N. The Antarctic bottom water can be traced as far as the Bay of Biscay.

Our knowledge of the speed of the water movements is small and based mainly on inferences from the temperature and salinity distribution. The problem of making direct measurements arises from the impossibility of fixing a current meter. Ships have been anchored as tightly as possible in deep water, but the over-riding, swinging, and yawing introduce so many errors that small currents cannot be measured. To measure the deep-water tides in this way is not impossible since they are generally faster than the non-tidal drift, and they can be extracted from the records because their periods are known.

A new method of measuring deep-water movements has been developed by Swallow [22] (1955). He fitted acoustic transmitters in aluminium tubes which are less compressible than sea water. If they are loaded at the surface they will sink until the increase in density of the water due to compression is sufficient to restore their buoyancy. Then as they transmit at regular intervals they can be followed with the help of two hydrophones mounted fore and aft in a ship. A float loaded so that it floated at a depth of 600 m was released in 41° N 14° W off the coast of Portugal and followed for 3 days. The tidal streams with speeds varying from 7 to 10 cm/sec made it follow a series of ellipses, but there was an average drift of 2·4 cm/sec (1·2 mile/day) towards 300° superimposed on the tidal streams. An increasing number of such measurements at varying depths is becoming available, ranging from 1 mile/day at 1,900 m between Madeira and the Canary Islands, to 4 miles/day at 1,200 m off the Straits of Gibraltar.

Although the deep currents are slow they carry large volumes of water and play an important part in the general circulation of the oceans. Stommel has suggested that the southward-flowing deep current in the Atlantic Ocean may intensify the Gulf Stream and weaken the Brazil current. They are rich in plant nutrients since they contain the products of decomposition of decaying animal life running down from the surface waters, and their movements and variations are responsible for regional differences and variations in the productivity of the ocean, and affect the fisheries. The work of the *Discovery II* in the Antarctic has shown a very close relation between the water circulation and the distribution of the marine plants and animals, particularly the whale food and the whales themselves. The deep movements transport large amounts of heat and are likely to affect the regional energy budgets and the quantity of heat transferred to the atmosphere.

Conclusion

I have not been able to do more than hint at many of the problems of marine physics, and have done practically nothing to show how they are related to the problems of marine biology, marine chemistry, and submarine geology. These are the more attractive aspects of oceanography. They infer the speed of water movements from C_{14} age determinations and the distribution of marine animals, and it is time they had some direct physical measurements to reckon with.

I cannot, after all, in so limited a space, give much idea how marine physics m[illegible] be useful to engineers, but I should like to plead that our experience at the Nation[illegible] Institute of Oceanography shows that a fairly systematic approach to its problem[illegible] often leads to the right man, method, instrument, or information being available [illegible] the right time when our assistance is sought with a wide range of marine problem[illegible] Finally, I ought to do justice to my colleagues by showing myself proud of the fa[illegible] that we do a lot of marine biology at the National Institute of Oceanography, a[illegible] by emphasizing that this is no less interesting and important than the physics.

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Sir Claude Inglis, in moving a vote of thanks to the Lecturer, said that what had struck him most about the Lecture was the extraordinary complexity of all the work which had had to be done by Dr Deacon and his staff, and the success which they had achieved in simplifying and resolving those highly complex problems.

Probably the easiest part of the Lecture for civil engineers to follow was the way in which, when two waves of equal amplitude but of slightly different periods were superimposed, they built up into high peaks and troughs when they synchronized, but cancelled out when the peak of one wave synchronized with the trough of the other. One had to reverse that process when the two waves had to be separated out from the resultant curve, and when, as in nature, the resultant curve comprised an extraordinarily complex wave train, the difficulty of separating out the various waves could be realized; yet Dr Deacon had succeeded in sorting out waves of all periods, forming a complex wave train. That achievement was of great importance to engineers. For instance, at the present time it was considered that "ranging" of ships in harbours was almost entirely due to very long waves. A wave of even 1 ft, if it was very long, would cause a ship to swing sideways, snapping its mooring chains, whereas a short wave, even though it were of large amplitude, had very little effect on "ranging"; because in that case the ship moved up and down but was not carried along sideways. The presence of long waves was not visible, and could only be determined by separating out the various wave lengths by analysis, and such an investigation was to be carried out shortly at a port on the west coast of Africa, to find out whether "ranging" of ships would be serious in the harbours on that coast; because in that case it would be necessary to ascertain, by model experiments, how to remedy the trouble by careful harbour design.

Another question which would be greatly clarified by wave analysis was coast erosion. Long waves reaching the shore caused the shore to build up, whereas short choppy waves combed down the loose material exposed on the coast; therefore all the information that could be obtained about the type of waves was important. In model experiments the correct types of waves had necessarily to be reproduced before the model could be used for designing works to prevent erosion by combing out the unsuitable waves. In the past, models had had to be designed by hit-and-miss methods, but owing to Dr Deacon's work it would now be possible to design models on a sound theoretical basis.

In conclusion, he was sorry to hear from Dr Deacon that people, like himself, who had to work in shallow water, got into much deeper water than those who worked in deep water!

He had great pleasure in proposing a vote of thanks to Dr Deacon.

Sir Arthur Whitaker, who seconded the vote of thanks, said that he had been struck by the great complexity of the problems that had to be solved. His life had been spent mainly in dealing with the sea and its effects, and he felt that some of the problems were still very intractable.

With regard to tidal range there was available a reasonable supply of data, but the sizes of waves were still very uncertain. He had recently been asked for advice at a place where the available data as to the size of waves had been negligible. One could only go back on the hearsay of the local fishermen who had had a limited period in which to observe, and who were often found to be very inaccurate. He was hoping that as a result of those researches the day would come when there would be a chart for the whole world which would give some idea of the maximum sizes of waves that could be encountered. Such charts were already available for temperature, tidal ranges, and rainfall, and he hoped that wave data would soon be similarly recorded.

In the invasion of Europe, there had been the case where, in the British port, the

placing of the ships that were the forerunners of the invasion had had to be done with gre accuracy. The timing had been known to a few minutes and there had been no indecisi there, but the gale which had come 3 weeks later had been to some extent unpredicted In the early days the design of the port had not taken into account waves that were know to exist. Fortunately, the remedy had been started and applied in sufficient time to sa the port from the risk of collapsing under heavy gales.

He heartily supported the motion for a vote of thanks to Dr Deacon.

The vote of thanks was accorded with acclamation, and the meeting terminated.

Paper No. 6136

NO-FINES CONCRETE AS A STRUCTURAL MATERIAL

by
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John Dixon Bolton, A.M.I.C.E., and
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(*Ordered by the Council to be published with written discussion*)

SYNOPSIS

No-fines concrete has for a number of reasons developed extensively in recent years as a material for building houses. Expanding production has encouraged research into the characteristics of the material and has led to advances in technique of building with it.

The Paper details these characteristics, the most important of which are specific weight, compressive strength, resistance to moisture penetration, and thermal and sound insulation values. The advantages provided by these characteristics, and also the limitations, dictate that economic architectural and structural design must proceed with full recognition of them; the best manner in which to utilize these characteristics is discussed.

Methods of construction adopted in Great Britain have usually involved the use of formwork constructed in very large units and have been accompanied by a high degree of mechanization. Methods of mixing and placing are described, also protection from the weather, and comparison is made with traditional methods of building in respect to economy and speed of construction. The material is especially advantageous where manpower is scarce or expensive.

Where the maximum potentialities of no-fines concrete in compression must be developed, as in tall buildings, careful mix design and control of quality is essential. It has been found that compressive strength bears a clear relation to relative specific weight, that the latter can vary considerably in practice, and that this can cause important variations in the compressive strength of walls. The degree of compaction in these circumstances is an important factor in control, and if capable of measurement can be used to interpolate the range of compressive strength obtained in practice.

INTRODUCTION

THE vast housing need in almost every European country created by the havoc of the 1939–45 war has stimulated the development of a number of new or previously little-used methods of building construction. Of these, one of the most successful methods, using no-fines concrete (or cellular concrete, as it is known on the Continent), has contributed substantially to the production of new houses, especially in Great Britain, Germany, and Holland, and also in France, Belgium, and Russia. In Germany, in particular, the extensive use of the method, and the considerable research undertaken into the properties of no-fines concrete, arose from the problem of disposing of large quantities of brick rubble and the possibilities of its use as aggregate when crushed and screened. Elsewhere, the temporary inability of the

* Mr Macintosh is Construction Manager, Mr Bolton is Senior Engineer, and Mr Muir is Research Assistant; all are with the Scottish Special Housing Association Ltd.

brick-making industry to meet the unprecedented demand led to its adoption, alon with other methods that minimized the use of bricks. Although this difficulty h been largely overcome, the no-fines-concrete method has proved competitive wi traditional building and continues to play an important part in the production houses, both at home and abroad. In Scotland, the additional motives provid by the lack of good facing bricks and the presence of unlimited supplies of ha aggregates have given added incentive to the use of this method, and no-fines-concre houses have made a considerable contribution to the total built since the war.

2. The method was introduced into Britain from Holland, the earliest examp being a group of 50 two-storey houses in Edinburgh completed in 1923, built wi clinker aggregate. Other clinker-aggregate houses were built at Liverpool, Ma chester, and Willesden. In the late 1930s the Scottish Special Housing Associati Limited, established in 1937 to relieve unemployment in Lanarkshire by buildi houses with the maximum proportion of unskilled labour, adopted the no-fine concrete method, using whinstone aggregate. By 1942, it had completed 901 hous in Lanarkshire, and at Rosyth and Dunfermline.

3. Subsequent to the war no-fines-concrete houses have been built by a number large building and civil engineering firms, including the Scottish Special Housi Association Limited.

4. With the expanded production in this material, many important technic advances have been made in its use. Pre-war production was confined to tw storey houses, but following the war three- and four-storey flats have been built c a large scale. The most ambitious project so far completed is the Max Käde Hau Stuttgart, having six lower storeys of dense concrete and thirteen storeys of no-fine concrete. The structure has no reinforcing frame, all no-fines-concrete walls bei load-bearing.

General description

5. No-fines concrete may be defined as a concrete from which the fine aggrega is almost, if not entirely, omitted. The aggregate is of a single size, usually ¾ in in Britain. The finished product is thus a cellular concrete of comparatively lo strength, which is partly compensated by reduced loadings resulting from its lo specific weight. The interstices form a barrier to moisture penetration by eliminati capillary attraction, and the cellular nature also provides greater thermal insulatio than exists in dense-concrete walls. It is common to the varying methods of con struction in use that it is poured in situ into formwork one-, two-, or three-storey in height, to form both external and interior partition walls, and it is normal for a window and door frames, beams, and other fixtures that are to form part of th walls to be fixed between the forms and the no-fines concrete to be poured aroun them. The method is usually associated with a comparatively high degree c mechanization. Fig. 4 (facing p. 690) shows a no-fines-concrete wall of good quality

Properties

Compressive strength

6. Tables 2, 3, and 4 give the results of tests made in the mobile laboratory of th Scottish Special Housing Association with various aggregates.* The gradings of th aggregates were as shown in Table 1. Each series of cube samples was made afte

* See Appendix.

initial tests had indicated the optimum water/cement ratio for each mix, this optimum being then adopted for each series. Fig. 1 shows typical curves from which optimum water/cement ratios were deduced.

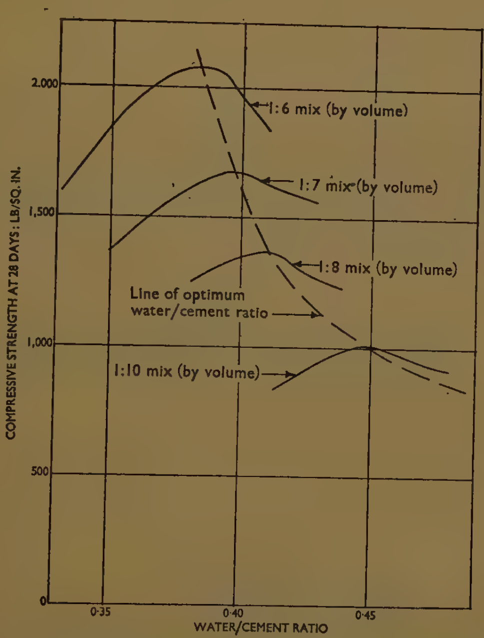

Fig. 1.—Variation in compressive strength of 6-in. test cubes of ¾-in.-gravel no-fines concrete for various mixes with variation in water/cement ratio

Table 1

B.S. sieve	Percentage by weight passing B.S. sieves		
	¾ in. whinstone	¾ in. gravel	¾ in. blast-furnace slag
1½ in.	100	100	100
¾ in.	80	94	100
½ in.	17	25	16
⅜ in.	1	1	2
3/16 in.	0	0	0

7. The laboratory test results show little difference in strengths obtained with gravel and with whinstone, but cubes made with blast-furnace slag are of a lower order. Against this, the lighter walls resulting from the use of slag give some compensation in reduced loadings. These strengths appear to be closely related to the specific surface areas of the aggregates, greater in the case of slag due to the

characteristic pitting that exists. The grading of the slag was also of shorter range than with the other aggregates.

8. The strengths that can be expected in walls, as distinct from cubes, are discussed later in this Paper. but experience shows that these will more nearly approach those obtained in the laboratory with cubes where the aggregate is rounded and free from flaky and elongated particles.

Thermal insulation

9. Very little research has been done on thermal insulation to date but it would appear from work done in Germany that thermal insulation is a function of concrete quality and relative density, the degree of thermal insulation depending upon the voids in the material. In Britain a wall thickness of 8 in. rendered externally and plastered internally is considered equivalent in insulation standard to an 11-in. brick wall with unventilated cavity and plastered internal finish.[1]

Sound insulation

10. The degree of sound insulation is determined primarily by the specific weight of the material; work done in Germany indicates that a reasonable standard of insulation can be attained provided the net weight of the wall is not less than 7[illegible] lb/sq. ft and the wall is plastered on both sides.

Fire hazard

11. A thickness of 10 in. is considered necessary to give adequate protection against fire.

Structural design

The basic concept and its influence on plan form

12. The successful utilization of no-fines concrete as a structural material depends entirely upon a complete understanding of its strength characteristics and especially a realistic appreciation of its limitations in this respect.

13. As a cladding material, its insulating and damp-resisting properties are dependent on the maintenance of a high degree of cellularity and whilst, by a careful selection of aggregate size and shape, this can be done without too great a loss of specific weight or density, there is obviously a limit to the density that can be attained without loss of cellularity. As discussed elsewhere in this Paper, strength is largely dependent on the density of the material and thus, in terms of maintenance of cellularity, the strength which can be achieved is limited. Also, by reason of its cellularity preventing the development of bond strength, no-fines concrete does not lend itself to the introduction of reinforcement to resist tensile stresses. The structural designer has thus to formulate his design on the basis of a material capable of resisting compressive stresses only and these of limited capacity.

14. To an even greater extent therefore than in conventional reinforced concrete design, it is necessary for the structural designer to collaborate with the architect in formulating the plan form of the building. Because of the limitations of his material the structural designer must see to it that, in his plan form, the architect has so disposed his walls as to ensure that the fullest possible use can be made of them as load-bearing members. Fig. 2 illustrates this point well.

15. In Fig. 2a the floor spans between front and back walls—thus necessitating

[1] "House construction." Post-War Building Study No. 1, H.M.S.O., 1944.

floor slab (in reinforced concrete) only 36% efficient in the ratio of super load to dead load, and with the end walls carrying none of the floor load there is a wall redundancy of 25%. By the introduction of cross-walls (Fig. 2b) to support the floor, the floor efficiency increases to 80% but the wall redundancy increases also (to 46%). By

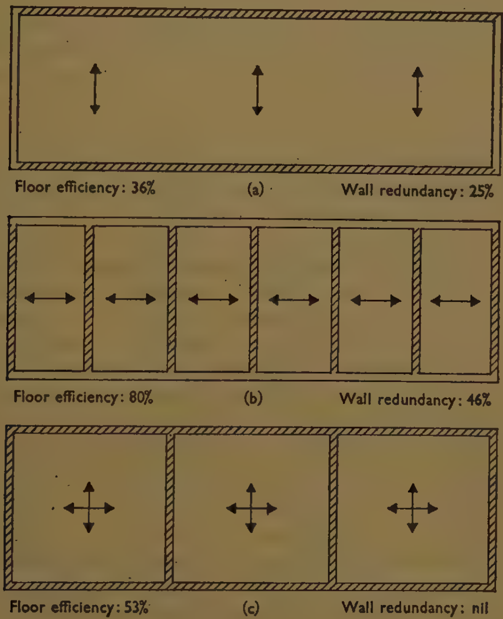

Fig. 2

the division of the overall shape of the building into square units as in Fig. 2c and the use of two-way spanning floor slabs the floor efficiency falls to 53%, but all walls become load bearing and redundancy is nil.

16. The basic concept for the structural design of a building in which no-fines concrete is to be used as a structural material and which materially affects the architectural plan form is, therefore, of a monolithic box form of structure all the walls of which are load bearing.

Walls

17. Not always, if indeed ever, can the structural designer hope to have his ideal of boxes, square or almost so, satisfied by the architect's plan form. It then becomes necessary to look at the problem in terms of wall stresses rather than total loadings and in this respect also there is an important characteristic of no-fines concrete to be borne in mind. Such research as has been done into the matter indicates that the modulus of elasticity E of the material is of a very low order. It is thus necessary, if cracking due to differential strain is to be avoided, to consider the incidence of wall stress, not only in individual but also adjacent walls, with the

object, so far as possible, of achieving uniformity of wall stress. Insofar as wa loads vary in terms of floor loads such uniformity will be achieved by variation : thickness of walls. Assessment of wall stress must also have regard for inciden of door and window openings and whilst the width of these may be allowed for, b making a suitable reduction in the length of wall under consideration, it is importan also to ensure that their placing in the wall does not create slender column sectio carrying high intensities of load. The minimum width of wall between two suc openings will, of course, depend upon the width of the openings themselves.

18. In the absence of a code of practice dealing specifically with no-fines concre as a structural material, the designer requiring guidance in the matter of permissib wall stress in relation to cube strength must have recourse to C.P. 111.201(194 which, although dealing with load-bearing walls of plain concrete, may, insofar : the two materials may be considered analogous in their strength characteristics, l considered valid for no-fines concrete. There is, however, in the opinion of th Authors, a good case for a separate code of practice for no-fines-concrete design an construction, having particular regard to the basic concept already outlined.

19. Whilst C.P. 111 does make some concessions in permissible wall stresses : relation to intersecting walls such concessions do not, in the Authors' opinion, ful reflect the advantages of monolithic box construction. This fact is well illustrate by a comparison of the provisions of the British and German codes of practice in th respect. Taking the limiting conditions laid down in the German code* for which load factor (cube strength/wall stress) of 5 is permitted (namely, a wall 11 ft 6 i high, 6 in. thick, with transverse walls not more than 26 ft apart), under the Britis code such conditions would enjoy no concession for intersecting walls and, makin allowance for the slenderness ratio, would require a load factor of 10. The fact tha buildings up to thirteen storeys in height have been built successfully in German with the lower load factor would appear to prove its validity for this form construction.

20. It is only in multi-storey buildings where wall stresses, multiplied by a appropriate load factor, come near to the cube-strength potentialities of the materi that wall loadings become the factor determining wall thickness. For dwellings u to five storeys in height, where wall loadings are moderate, wall thicknesses a determined by other considerations such as thermal and sound insulation and fi resistance (§ 9–11). Where such is the case the mix and strength of the no-fine concrete can be reduced and there is also not the same need for close observance the basic concept of square plan form with two-way spanning floor slabs. Becau of this, great economies in construction can be achieved by the use of a cross-wa form of plan in conjunction with either precast concrete units or conventional timbe joist floors spanning in one direction only.

Floors

21. Although not themselves constructed of no-fines concrete, floors play a important part in the design concept to the extent that they contribute to the overa stiffness and stability of the building.

22. The form of connexion and degree of fixity between floor and wall have a important bearing on the height of wall, on which slenderness ratio and permissib wall stress are determined. Where, as in the case of four- or five-storey dwelling wall loadings are moderate and wall thicknesses, for other reasons, lead to low wa

* DIN 4232. Geschüttete Leichbetonwände für Wohn-und andere Aufenthaltsraüm Richtlinien für die Ausführung.

TABLE 2.—TESTS USING $\frac{3}{4}$-IN. GRAVEL AGGREGATE
(Loose specific weight: 94·5 lb/cu. ft)

Mix proportion by volume	Water/cement ratio	Age at test: days	Specific weight at stripping: lb/cu. ft	Specific weight at crushing — Mean: lb/cu. ft	Specific weight at crushing — Standard deviation: lb/cu. ft	Specific weight at crushing — Coefficient of variation: %	Cement content: lb/cu. yd	Ultimate compressive stress — Mean: lb/sq. in.	Ultimate compressive stress — Standard deviation: lb/sq. in.	Ultimate compressive stress — Coefficient of variation: %	Percentage of 28-day strength
1:6	0·383	3	126·4	125·8	0·62	0·49	436	1,295	35·9	2·75	62·2
1:6	0·383	7	126·3	125·4	0·46	0·37	435	1,662	97·1	5·85	79·8
1:6	0·383	28	126·5	124·8	0·12	0·10	436	2,080	105·1	4·94	100
1:7	0·397	3	123·1	122·6	0·80	0·65	373	1,042	18·7	1·96	62·0
1:7	0·397	7	123·2	122·3	0·17	0·14	373	1,292	16·9	1·22	77·0
1:7	0·397	28	123·1	121·8	0·87	0·71	373	1,680	131·1	7·5	100
1:8	0·41	3	120·4	120·0	0·92	0·77	325	848	23·8	2·84	62·1
1:8	0·41	7	120·6	119·5	0·78	0·65	326	1,053	81·3	7·12	77·1
1:8	0·41	28	120·8	119·4	0·76	0·63	326	1,366	35·4	2·59	100
1:8	0·41	91	120·7	119·4	0·65	0·54	326	1,660	184·4	15·8	121·6
1:10	0·447	3	117·2	116·7	0·75	0·64	261	624	24·8	4·03	61·5
1:10	0·447	7	117·1	116·4	0·35	0·30	260	780	54·7	6·94	76·8
1:10	0·447	28	117·3	116·2	0·46	0·40	261	1,015	95·8	9·22	100

TABLE 3.—TESTS USING $\frac{3}{4}$-IN. WHINSTONE AGGREGATE
(Loose specific weight: 99 lb/cu. ft)

Mix proportion by volume	Water/cement ratio	Age at test: days	Specific weight at stripping: lb/cu. ft	Specific weight at crushing			Cement content: lb/cu. yd	Ultimate compressive stress			Percentage of 28-day strength
				Mean: lb/cu. ft	Standard deviation: lb/cu. ft	Coefficient of variation: %		Mean: lb/sq. in.	Standard deviation: lb/sq. in.	Coefficient of variation: %	
1:6	0·333	3	129·2	128·8	0·36	0·28	432	1,192	64·3	5·4	55·3
1:6	0·333	7	129·3	128·8	0·42	0·33	432	1,659	113·0	6·78	77·0
1:6	0·333	28	129·4	128·6	0·2	0·16	433	2,153	31·9	1·48	100
1:7	0·338	3	127·0	126·6	0·54	0·43	372	1,015	29·4	2·9	56·3
1:7	0·338	7	126·9	126·4	0·76	0·60	372	1,370	48·5	3·54	76·0
1:7	0·338	28	127·1	126·2	0·23	0·18	372	1,801	63·5	3·52	100
1:8	0·348	3	124·4	124·1	0·67	0·54	324	808	30·7	3·8	55·3
1:8	0·348	7	124·5	124·1	0·32	0·26	324	1,108	10·0	0·90	75·8
1:8	0·348	28	124·6	123·8	0·7	0·56	324	1,461	42·7	2·92	100
1:8	0·348	91	124·4	124·5	0·57	0·46	324	1,897	117·6	6·2	130·0
1:10	0·36	3	121·5	121·2	0·43	0·35	260	586	35·3	6·02	56·5
1:10	0·36	7	121·6	121·2	0·41	0·34	260	798	31·5	3·95	76·9
1:10	0·36	28	121·7	120·7	0·45	0·37	260	1,038	38·2	3·68	100
1:12	0·372	3	119·7	119·3	0·53	0·44	218	452	36·3	8·03	58·4
1:12	0·372	7	119·6	119·1	0·66	0·55	218	588	30·2	5·14	76·1
1:12	0·372	28	119·7	118·9	0·69	0·58	218	773	46·1	5·97	100
1:15	0·392	3	117·6	117·2	0·47	0·40	174	299	27·1	9·07	57·5
1:15	0·392	7	117·6	117·1	0·61	0·52	174	395	36·2	9·17	76·0
1:15	0·392	28	117·7	117·0	0·51	0·44	174	520	51·9	9·99	100

TABLE 4.—TESTS USING $\frac{3}{4}$-IN. AIR-COOLED BLAST-FURNACE-SLAG AGGREGATE
(Loose specific weight: 79·4 lb/cu. ft)

Mix proportion by volume	Water cement ratio	Age at test: days	Specific weight at stripping: lb/cu. ft	Specific weight at crushing			Cement content: lb/cu. yd	Ultimate compressive stress			Percentage of 28-day strength
				Mean: lb/cu. ft	Standard deviation: lb/cu. ft	Coefficient of variation: %		Mean: lb/sq. in.	Standard deviation: lb/sq. in.	Coefficient of variation: %	
1:6	0·335	3	110·4	109·9	1·17	1·07	441	1,151	62·6	5·43	65·2
1:6	0·335	7	110·5	109·9	1·32	1·2	441	1,416	145·7	10·3	80·3
1:6	0·335	28	110·5	110·0	1·2	1·09	441	1,763	53·1	3·01	100
1:7	0·345	3	107·2	106·9	0·76	0·71	377	888	66·0	7·43	62·1
1:7	0·345	7	107·2	106·8	0·58	0·54	377	1,138	64·3	5·65	79·4
1:7	0·345	28	107·3	106·9	1·32	1·23	378	1,432	75·9	5·3	100
1:8	0·355	3	105·2	104·8	0·36	0·34	331	698	45·5	6·52	58·4
1:8	0·355	7	105·4	105·0	0·47	0·45	332	944	33·1	3·51	78·9
1:8	0·355	28	105·5	105·1	0·12	0·11	332	1,196	70·8	5·93	100
1:8	0·355	91	105·4	104·9	0·71	0·68	332	1,407	89·4	6·35	117·6
1:10	0·37	3	102·6	102·3	0·75	0·73	266	467	22·6	4·83	57·5
1:10	0·37	7	102·7	102·3	0·91	0·89	266	650	24·8	3·82	80·1
1:10	0·37	28	102·6	102·3	0·55	0·54	266	812	39·5	4·86	100
1:12	0·388	3	100·8	100·5	0·63	0·63	222	336	23·7	7·06	55·1
1:12	0·388	7	100·7	100·4	0·77	0·77	222	451	20·3	4·5	74·0
1:12	0·388	28	101·0	100·7	0·54	0·54	223	610	47·6	7·8	100
1:15	0·413	3	98·3	98·0	0·20	0·20	177	191	19·7	10·3	54
1:15	0·413	7	98·3	98·0	0·49	0·50	177	262	20·3	7·75	74
1:15	0·413	28	98·4	98·1	0·37	0·38	178	354	34·7	9·8	100

stresses, the degree of fixity between floors and walls may be considered adequa using either one-way spanning precast floor units bearing directly on the wall, timber joists connected by steel angle cleats at every third joist to a timber runn fixed to the wall by bolts shrouded in precast dense-concrete core blocks in the wa

23. For multi-storey dwellings, however, where wall stresses are more cruci calling for a square plan form in association with two-way spanning in-situ concre floor slabs, a more effective form of connexion is achieved by the introduction horizontal ring-anchor bars in the edge of the floor slab, connected through vertic dowel bars with another two ring-anchor bars in the walls themselves, one abo and one below the floor slab. This form of construction not only provides positi fixity of floor to wall, thus reducing the slenderness ratio and allowing a high permissible wall stress, but also acts very effectively to give lateral stiffness to t box form of construction to resist the increased horizontal wind loadings associat with high buildings.

Foundations

24. In no-fines concrete, as in other systems of construction, the aim of t engineer is to equalize so far as possible loading of the subsoil and he is assisted this respect by the box form of construction characteristic of the no-fines metho He must, however, appreciate the strength limitations of his material and reali that the superstructure will offer little or no resistance to the stresses set up possible differential settlement.

25. A subsoil uniformly consistent in elastic properties is very improbable, an such inconsistencies as there may be must be revealed by a thorough exploratio and sampling of the subsoil over the area of the building. With this information careful assessment of probable settlement will require to be made and the foundatio designed accordingly.

26. Differential movement in the superstructure being impermissible the founda tion must provide the maximum rigidity, and whilst three- and four-storey building in no-fines concrete have been successfully built on conventional strip footings, fo higher structures it is usual to resort to a box raft form of foundation constructed i conventional reinforced concrete. Such a form of construction is obviously cost and thus can only be justified in terms of cost per unit dwelling by multi-store construction.

27. As already discussed in respect to the economic disposition of floor loads o walls, the need for close collaboration between architect and engineer is also essenti in regard to the influence of plan form of the complete building on foundation loading The effect of wind load on the long rectangular or, as it is sometimes called, "slab building can give rise to embarrassing inequalities of soil loading and experienc shows that the "point" block, of approximately T or cruciform shape, with a secon moment of area more nearly uniform about any axis which may be selected relativ to wind direction, helps to achieve the uniformity of loading desired.

Methods of construction

Formwork

28. Many varying types of formwork are in use for the construction of no-fin concrete walls. On the Continent the normal structural design provides for eac floor to be of in-situ reinforced concrete, and forms are in consequence normall one storey in height. One successful system used in Germany consists of a series forms of varying widths, based on a 62·5-cm module, comprising woven wire clot

welded to a frame of pressed steel angles. They are of such sizes that all are readily manhandled and can be adapted easily to different layouts. Systems using timber-boarded forms, with various means of framing, are also common in Germany and especially in Holland.

29. In Great Britain the most successful methods, for buildings up to four storeys in height, have been based upon forms either two or three storeys in height, or made up of large panels of storey height assembled for use as two- or three-storey forms. These initially consisted of timber frames covered with either expanded metal or woven wire cloth, inner and outer forms being connected with through bolts. With forms of these dimensions handling was by means of cranes, which were also used for pouring the concrete. Compared with small multi-purpose forms these systems reduced considerably the time taken in erecting and straightening the forms, and also in dismantling. They had the advantage (very necessary when pouring two or three storeys at a time) of enabling observation to be made of the filling of the forms and of ensuring that no cavities were left. Their use depended upon flooring systems which allowed the floors to be placed in position after the walls had been poured. In addition, owing to the large sizes adopted and the resulting limited flexibility, their use was based upon the production of houses built to standard layouts.

30. Earlier results from these forms were inferior in quality to those obtained on the Continent owing to the tendency of the expanded metal or wire cloth to bulge after repeated uses, giving extremely uneven wall surface. This necessitated frequent refacing of the forms and costs were, in the Authors' experience, of the order of £37 10s per house for the capital cost of the forms and their maintenance. One large contracting firm has largely overcome this trouble by stiffening up the facing by a backing of welded mesh.

31. Another type of form widely used in Scotland, which has been very successful, consists entirely of steel, the facing being 14-gauge perforated sheet welded to a steel frame. The largest panel is 8 ft $4\frac{3}{4}$ in. high × 6 ft $8\frac{3}{4}$ in. broad; four basic widths of panel are found to be adequate, with special triangular forms for gable peaks. The panels can be bolted securely together, two or three storeys in height, stiffened with horizontal and vertical steel walings as shown in Fig. 5. These forms are designed for a maximum horizontal pressure of 75 lb/sq. ft, on the assumption that no-fines concrete exerts roughly one-third of that exerted by dense concrete.

32. Certain of the internal forms of this type are provided with a row of hinged gates arranged at window-sill level; these can be dropped at window openings to enable the completion by hand of the pouring below the window frames.

33. The steel forms have been found to have a great many advantages over the timber-framed form, despite the greater initial capital cost. These can be listed as follows:—

(*a*) Provided building programmes permit continued use of forms of the dimensions adopted, their life seems almost indefinite.

(*b*) Maintenance is only necessary in the event of accidents; experience shows that these are rare and that buckling is far less frequent than with small forms that can be manhandled, and, in consequence, more easily misused.

(*c*) The smooth face of the forms results in far less adhesion of cement than with expanded metal or wire cloth, and the face is easily cleaned with a flat scraper.

(*d*) A good surface is given for the retention of mould oil.

(*e*) The rigid perforated steel sheets provide, in addition to visibility of the pour concrete, fixing points wherever required for bolts and cleats holding position door and window frames, ventilators, and other fixtures. (S Fig. 6.)

(*f*) The true face left after striking provides the basis for application of finis without the necessity of preliminary trueing-up.

(*g*) Given a reasonable size of building programme, the capital and maintenan charges per house are much smaller than for timber-framed forms.

34. The weight to be handled differs little from that of timber-framed formwo this being 9·3 lb/sq. ft for steel, and 7½ to 8 lb/sq. ft for timber-framed forms, includi walings. Striking of forms is normally carried out on the day following pourin but in cold weather longer time may be necessary.

35. Table 5 shows typical outputs attained in oiling, erecting, striking, and clean the steel forms described, including fixing door and window frames, lintels and oth fixtures, on three- and four-storey flat construction.

TABLE 5

Unit	Per sq. yd	Per house
Man-hours	0·33–0·44	86–106
Crane-hours	0·06	14·3

36. The higher figures of man-hours given are for four-storey construction carri out in two lifts. The lower figures are for three-storey construction in one lift.

Mixing

37. In the Authors' experience, close control of quality in mixing no-fines concre is worth while in enabling leaner mixes to be used to achieve given minimum specifi strengths. Careful control of water content is of the greatest importance; the u of wet mixes, in particular, is to be avoided, and the aim should be to provide ea particle of aggregate with a coating of cement mortar that adheres to it duri placing. Any tendency for the mortar to run can lead to the formation of den patches that can result in moisture penetration. A mixing time of 1 min is foun to be adequate and this can be given by operating on a cycle of 2 min, allowing tim of loading and discharge.

Placing

38. Pouring should be in a series of horizontal layers working in a circular fashi round the portion of the building being poured, and not pyramid fashion leavi sloping faces to the concrete which may be disturbed by subsequent batches plac after initial set has occurred, thus creating planes of weakness. Great care necessary to ensure, by rodding, complete filling of the forms under the ends of bean and lintels and around fixtures, and provision is necessary in the design of the form to enable pouring below windows to be completed by hand. Up to four storeys height it has not been found necessary to give any compaction other than th obtained by the drop from the crane skip, but for buildings of greater height, wh the no-fines concrete walls are load bearing and strengths more critical, compacti

is necessary. Table 6 shows typical outputs attained in the operations of mixing and placing no-fines concrete in the walls of three- and four-storey flats.

TABLE 6

Unit	Per sq. yd in walls 12 in. thick	Per house
Man-hours	0·3	31
Crane-hours	0·03	3·1

Protection from severe weather conditions

39. Owing to its cellular nature, no-fines concrete is not subject to the exertion of internal pressures under icing conditions. Mixing and placing can proceed in several degrees of frost provided steps are taken to enable hydration of the cement to proceed at a reasonable rate. The concentration of the pouring of walls, two or three storeys at a time, within a fairly confined area, facilitates greatly the precautions necessary. The addition of calcium chloride to the mix, combined with steam heating of the aggregate in the batching hopper and the use of coke braziers in the bays being poured, has enabled work to proceed without interruption in 5° of frost.

40. A far more dangerous condition, arising from the cellular nature of the concrete and the open type of formwork customary, is the combination of a warm sun and a fresh drying wind. Unless freshly poured walls are covered and damped down, this condition can result in loss of water by evaporation throughout the thickness of the wall before setting has taken place. The use of calcium chloride to accelerate setting has also been used under these circumstances with success. A few actual failures are known to the Authors when these precautions have not been taken, varying from disintegration of the outer face of the wall on the windward side, to a case, in the summer's drought of 1955, of complete disintegration of 12-in. walls two-storeys high poured the previous day. The conditions under which these failures are possible are comparatively rare in the British climate, and the builders are in consequence more liable to be caught unawares than in a drier climate where these precautions would become a necessary routine.

Finishes

41. No-fines concrete requires the application externally of a treatment to fulfil the dual purposes of forming a barrier to the penetration of rain, and of decoration. This is normally done by roughcasting, and it is important that the mix used gives the least tendency to crazing or hair-cracking. No-fines concrete provides a key that it would be difficult to match in any material.

COMPARISON WITH TRADITIONAL BUILDING METHODS

42. Comparison between building methods on the basis of cost, speed of construction, or of manpower expended, is extremely difficult to make with precision, since it is rarely, if ever, possible to compare like with like. The Authors prefer, therefore, where figures are quoted, to leave it to those interested to make such comparisons as they can from their own experience.

43. The no-fines-concrete method enables the introduction of an extremely high degree of mechanization to house building in respect of the building of the walls.

Thereafter the later operations tend to follow traditional methods, although some them may be assisted by the lifting equipment present. In regard to cost it do appear that, in Britain, it is competitive with brickwork where circumstances favourable to the method. Such favourable circumstances are those of large si where the number of different house types are limited, and preferably related each other in such a way as to minimize the production of forms of special siz Under circumstances where the relative cost of skilled labour to that of machine higher than at present exists in Britain, any economic advantage that lies with t no-fines-concrete method becomes enhanced.

44. There seems no doubt that the method considerably reduces the time norma taken in the building of the house shell, and that it is far less subject to interrupti by severe weather conditions. The only weather conditions that seriously hamp operations are those of high winds, which may immobilize cranes and prevent t movement of large sections of formwork. These conditions are comparatively ra and are not usually of long duration. Apart from the advantages to the operati of steady employment, the comparative absence of interruptions to progress on th essential element of the building reduces greatly one of the main disparities building work as compared with factory production, and does in fact enable t rhythm of factory production, with the constant co-ordination of all the vario processes, to be closely paralleled. Production, measured by numbers of hous according to type, is simply a factor of the number of cranes used, assuming that other items of plant and equipment are properly proportioned. This factor can taken as four houses per crane per normal working week for two-storey cottag and three for three- and four-storey flats. In addition, roofing can proceed at much earlier date, giving weather protection and speedy entry of the finishing trad This is illustrated in Fig. 7, where the shuttering and pouring of each six-hou block represents one week's work.

45. In remote areas, where ample supplies of aggregate are available local haulage of materials for the walls over long distances is confined to that of the delive and final removal of plant and formwork, and the delivery of the cement. Th compared with brick houses, the tonnage of materials to be hauled long distanc would be greatly reduced, and such circumstances would be particularly favoura to the use of no-fines concrete for housing.

46. With regard to manpower, the following are actual examples from contra completed for no-fines concrete houses:—

(*a*) 246 two-storey houses, 50% two-bedroom, 50% three-bedroom, avera floor area 760 sq. ft. Average labour time per house for all trad including underbuilding and site works, 1,315 man-hours. Average tim to complete each house, 26 weeks.

(*b*) 78 three-storey flats, comprising 52 two-bedroom and 26 three-bedroo flats, average floor area per flat 709 sq. ft. Average labour time for pouri exterior walls and cross-walls, all 12 in. thick, 123 man-hours/flat or 1· man-hours/sq. yd measured across openings. (Crane-hours were 17·4 p house, or 0·163 per sq. yd.) Average labour time per flat for all trad including underbuilding and site works, 1,576 man-hours.

47. The method is not well suited to the small builder, with limited capital, owi to the high initial cost of plant and formwork. It is also less flexible than metho based upon the use of small building units, and is not suited to small isolated sites.

Fig. 4.—Example of no-fines-concrete wall of good quality

Fig. 5.—Six-panel shutter assembly being lifted into position

FIG. 6.—VIEW OF STEEL SHUTTERING WITH WINDOW FRAMES AND PROFILES IN POSITION

FIG. 7.—THREE-STOREY NO-FINES CONCRETE FLATS UNDER CONSTRUCTION

Mix design and control of quality

48. Except with two-storey houses, whose wall thicknesses will be determined solely by the requirements of thermal insulation and which will be understressed even with a 1 : 10 mix, greatest economy can be obtained by the specification of minimum compressive strengths. Specifications based on mix proportions take no account of the control of quality that will obtain, and may result in local weaknesses in the structure owing to the absence of incentive to tight control that is given by leaving the mix design to a competent builder.

49. Mix design and control of quality are closely interrelated. The compressive strengths obtained in the laboratory with various aggregates, as given in Tables 2, 3, and 4, cannot be applied to mix design without knowledge, gained from experience, of their relation to the mean strengths obtainable with similar mixes in the field, and also the relation between minimum strengths and mean strengths for site cubes. The factors causing variation in cube-test results are fewer in number than with dense concrete because of the omission of the fine aggregate, in the same way as the introduction of gap-graded concrete produces closer control. For this reason, bulking variations are absent, water/cement ratios are more constant, and proportioning can be more accurate.

50. Tests carried out by the Authors indicate that with identical aggregates and weigh-batching of the cement a coefficient of variation of 12% can be attained on sites.

Relation of wall strengths to cube strengths

51. Test cubes, although serving as a check on the quality of the batches of concrete emerging from the mixer, give little indication of the actual strength of the concrete as placed in the walls. Whilst this is true of normal dense concrete it is even more true of no-fines concrete, which, being poured in shutters of minimum height of one storey, normally receives very little compaction. In order to obtain more information on the importance of this factor, tests have been carried out relating compressive strengths of 6-in. cubes at 28 days to specific weights. The results, as will be seen from Fig. 3, fell within a fairly well-defined envelope and, on comparing the envelope obtained for different mix proportions but using the same aggregate, it was found that they were continuous, with some overlap between the results of the various mix proportions. This shows clearly that degree of compaction is a factor of the highest importance in relation to compressive strength. It will be noted that some results with weaker mixes are stronger than some with richer mixes, due entirely to greater compaction in these cases. It would appear from this that, irrespective of the proportion of cement in the mix, if it can be determined that the specific weight of the no-fines-concrete wall is of a certain value it can be deduced that the strength of a 6-in. cube cut from that wall is likely to fall within a certain well-defined range. In applying this it is necessary to make allowance for the fact that measurement of the density of a wall as a whole gives only a mean value, and within the wall some variation will exist.

52. Measurement of mean wall densities was made on a site where no compaction was attempted by calculations based upon the number of batches that were used in each complete day's pour, coupled with careful measurement of the walls poured. The mix was 1 : 8 by volume and the whinstone aggregate used contained a proportion of flaky and elongated particles, which had a specific weight after rodding 13%

greater than the loose specific weight. Mean specific weights of the walls vari from 87·3% to 92·8% of those of the control cubes.

53. Similar tests were carried out on another building site, where a more round gravel aggregate was in use. The increase over the loose specific weight obtain

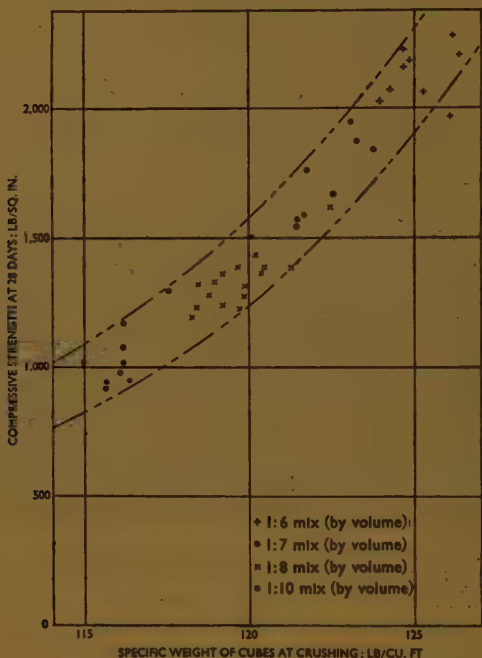

FIG. 3.—VARIATION IN COMPRESSIVE STRENGTH OF 6-IN. TEST CUBES OF ¾-IN.-GRAV NO-FINES CONCRETE OF VARYING MIX PROPORTIONS WITH VARIATION IN SPEC WEIGHT OF CUBES AT CRUSHING

(The continuous envelope is indicated)

by rodding was, in this case, only 6·4%. The lower lift of two storeys was be poured with a 1 : 8 mix whilst the upper lift was 1 : 10; thus it was possible to wid the scope of the measurements. In addition, a series of control panels, 24 in. hi × 18 in. wide × 12 in. thick, were cast between steel forms similar to those in use the flats and, for each control panel, three 6-in. cubes cast. In order to achiev degree of compaction nearer to that of the cubes the forms on the portions of flats being poured, and on the control panels, were tapped on both sides simult eously with rubber-headed hammers (a method commonly in use in Holland timber-boarded forms). Walls and panels averaged in specific weight 96·3% 97·3% the mean of the cubes respectively. It was possible to weigh the pan instead of relying on calculations as for the walls.

54. The attempt was made to carry the comparison of wall panels and cu further by means of crushing tests, but at this stage the results proved inconclus owing to inexperience in the technique of crushing large specimens, and failure recognize the magnification of testing errors that can occur when so doing.

55. Subsequent casting of panels similar in dimensions and crushing under better-controlled conditions gave compressive-strength results which, when plotted against specific weight, fell within the lower part of the envelope containing test results of cubes made of similar aggregate and mixed under similar conditions. These tests require to be extended, particularly in the range of specific weight covered, before they can be regarded as conclusive. Lower mean results for panels than for cubes can, however, be expected to arise from the following causes, quite apart from the differing dimensional characteristics themselves (the reduction factor for slenderness ratio, in accordance with C.P. 111, would be 0·97 only):—

(a) Greater variation in density in the larger samples.
(b) Difficulty in obtaining exact parallelism in the larger specimens.
(c) Slower rates of loading per unit area.

56. In the most recent tests carried out by the Building Research Station on larger wall panels of no-fines concrete, three wall panels were cast, each 9 ft high × 4 ft 6 in. wide × 10 in. thick, composed of no-fines concrete of a 1 : 8 volume mix, using $\frac{3}{4}$-in. to $\frac{3}{8}$-in. uncrushed river-gravel aggregate. A large number of control cubes were cast at the same time. The walls were tested to destruction in a 500-ton machine at ages of 33, 34, and 36 days respectively, corresponding cubes being tested at the same ages. Both walls and cubes gave very consistent results, and the crushing strengths of the walls ranged from 0·50 to 0·54 of the mean strengths of the cubes. The specific weights of the walls were estimated to be 97% of those of the cubes.

57. Allowing for the expected drop in strength due to this lower specific weight, cubes of the same specific weight as the wall would have failed at about 900 lb/sq. in. compared with the actual mean of 1,280 lb/sq. in. Compared with the mean wall strength of 657 lb/sq. in. this would have given a ratio of strengths of 0·73 wall to cube. The reduction factor for a wall of these dimensions due to its slenderness ratio would be 0·706 (based on C.P. 111), a figure remarkably close to the ratio of wall to cube strengths after correcting for the reduced specific weight. Design in Germany has been based upon the assumption that walls of storey height will possess 0·5 times the strength of cubes, which appears to be a reasonable approximation of the combined effect of slenderness ratio and of a specific weight in walls about 3% less than that obtained in cubes.

58. Sufficient has been said to demonstrate the paramount importance of obtaining satisfactory wall densities where the use of the full potential strength of no-fines concrete is required. When sufficient data have been amassed from compression tests of cubes made with a standard aggregate and varying mix proportions reasonably accurate forecasts of wall strengths can be assessed provided the wall densities can be measured. When this stage has been reached in any large building project the making of further cubes lessens in importance for control, and the really important factor to control is that of compaction. A rational method of wall design would be based upon load factors that took into account the relative specific weight to be achieved in construction, as well as minimum cube strengths and dimensional characteristics.

Conclusions

59. No fines concrete construction requires considerable initial capital investment, skilful structural design, and a highly developed technique and control. It is admirably suited to the requirements of a large building organization able to build large numbers of houses of a limited range of plans, preferably designed by the firm itself to suit the technique. Where the latter condition is fulfilled large steel forms can be used with advantages of long life and ease of erection, as compared with

multi-purpose forms in small units, with much higher erection costs, or more temporary timber forms.

60. The resulting houses are warm, dry, substantial, of long life, quick to erect and economical in cost. The economies in cost are greatest where suitable aggregate is readily available, where bricks or alternative walling materials are scarce or costly, and where skilled building labour is scarce or costly.

61. The properties of no-fines concrete are all closely related to the relative densities obtained in practice, and data concerning compressive strength, heat, or sound insulation are lacking in meaning unless tied to relative density.

62. The drafting and issue of a code of practice for no-fines-concrete construction is overdue. Such a code should, for the higher range of stresses that would be met in tall buildings, specify minimum cube strengths to be obtained rather than the mix proportions and should standardize methods of cube manufacture and testing procedure. It is also suggested that such a code should, again, for the higher range of stresses, specify the attainment of a defined relation between wall and cube densities, as a measure of the degree of compaction obtained in practice and, thus of the compressive strengths of the walls of the structure. The development of such methods of measurement as those based upon the transmission of supersonic frequency vibrations or of nuclear particles would undoubtedly make this much easier to apply.

Acknowledgements

63. The Authors have been able to present in this Paper the results of research carried out under the auspices of the Scottish Special Housing Association Limited and desire to record their thanks to its Council of Management and their General Manager, Mr S. A. Findlay, O.B.E., for permission to do so. They are also indebted to Mr G. Ross, Secretary of the Association, for his help in providing historical data. They thank the Director of the Building Research Station for making available to them the details of their full-scale tests of wall panels.

APPENDIX

METHOD OF MAKING AND TESTING NO-FINES-CONCRETE CUBES

64. All samples were mixed in a rotating horizontal pan-type mixer with paddles revolving in the same direction as the pan. Six-in. cubes were cast in steel moulds, but, for a number of reasons, the method of making the cubes did not comply with that specified in B.S. 1881 : 1952. It was found initially that it was difficult to make a uniform cube by the standard method owing to the granular nature of the material, and it was also considered that fully compacted cubes would bear little relation to walls, in casting which little or no compaction was given other than that caused by the drop from the skip. The aim was set, therefore, of obtaining cube samples of uniform no-fines concrete with little compaction by placing the material in the mould a trowel-full at a time, taking great care that corners were filled and good test faces produced by poking the coated aggregate with the trowel and working the latter up and down the sides of the mould. This method has produced cubes of good consistency and little variation in any series from the mean compressive strength and specific weight. Cubes were stored under damp sacks at a temperature of 60°F. Each series of cube samples was made after initial tests had indicated the optimum water/cement ratio for each mix, this optimum being then adopted for each series. Compression tests were made in all respects in accordance with the method specified in B.S. 1881 : 1952.

The Paper, which was received on 3 July, 1956, is accompanied by four photographs and three sheets of diagrams, from which the half-tone page plates and the Figures in the text have been prepared, and by an Appendix.

Correspondence on this Paper should be forwarded to reach the Institution before 15 February, 1956. Contributions should not exceed 1,200 words.—Sec.

Paper No. 6137

HEAT FLOW IN THE STEAM CURING OF CONCRETE PRODUCTS

by

*** Professor Allan Dawson Ross, Ph.D., B.Sc., M.I.C.E.**

(*Ordered by the Council to be published with written discussion*)

SYNOPSIS

The beneficial effects of steam curing on the growth of strength of a concrete product might appear to diminish with increasing size of the product. The Paper describes an investigation which has shown that although the internal temperature/time and temperature/space gradients differ widely, the integral of temperature and time for a particular treatment actually increases somewhat with increasing size. The heat of hydration of ordinary Portland cement accelerates the rise in temperature but its effects are secondary in importance.

INTRODUCTION

Steam curing is widely employed to accelerate the growth in strength of concrete products because it allows the early release of moulds and pre-tensioning plant, and reduces the storage space otherwise required for normal curing. The growth in the strength of a particular concrete depends not on its age, but on the integral[1] of time and temperature and this parameter, measured in °C-hours, is referred to as the "maturity".[2] This statement is not exact since the temperature/time gradient and other factors are also influential. However, although strength is related to temperature and time in some more complicated manner than the simple integral, it is sufficient to note that it is a function of the temperature history.

The compressive strength is assessed by crushing cubes which, since they are heat cured identically with the product, are assumed to have the same maturity and, therefore, the same strength. The actual product, however, differs both in size and shape from the cube and therefore its temperature history will be different. For example, if the surface temperatures of two cubes, one 20 in. and one 4 in., are raised identically by steaming, the temperature at the centre of the larger cube will rise at one twenty-fifth of the rate of the smaller cube, the time scale varying with the square of the linear dimension. The maturities achieved during heating are therefore quite different in the two cases.

Furthermore, the temperature history varies from point to point inside the same specimen. Fig. 1 shows the computed temperature/time curves for two points in a long block of concrete subjected to an arbitrary variation of surface temperature. Although the temperature of the surface is raised 60°C the central point experiences

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[1] The references are given on p. 701.

a rise of only 37°C and this occurs 1½ hours after cooling has commenced, thu emphasizing the time lag and the possibility of a low temperature in the interior.

Superficially, therefore, it would seem that the effects of steam curing varied wit the size of the specimen and perhaps also with the position of points within the sam specimen. To investigate this matter tests were conducted in a curing chamber

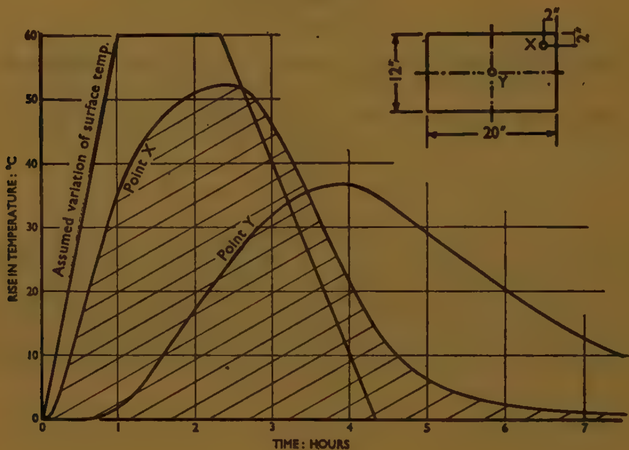

FIG. 1.—EXAMPLE OF TWO-DIMENSIONAL FLOW OF HEAT WITH A DIFFUSIVITY OF 0·1 SQ. IN./MIN

16-cu. ft capacity supplied with steam at atmospheric pressure. Temperatures i the chamber and inside the specimens were measured by copper-constanan therm couples connected through a twenty-way switch to a suitable indicator. Th concrete was a 3 : 1½ : 1 nominal mix with ordinary Portland cement and a wate cement ratio of 0·45.

THE TESTS

Value of the diffusivity

The preliminary tests were designed to determine the value of the thermal diffus vity. Cylinders, of 6 in. dia., with heavily insulated ends were used to ensure tha the flow of heat was two-dimensional. Nine thermo-couples were cast in eac cylinder, three distributed along the central axis, three at a radius of 1½ in., and thre at a radius of 2½ in. The three thermo-couples within each group gave practicall identical readings, thus confirming the two-dimensional conditions.

A typical result is shown in Fig. 2 for a cylinder which was immersed in steam days after casting. The interior temperatures were computed from the exterio temperatures using a diffusivity of 0·1 sq. in/min, and the agreement between th experimental and calculated results in Fig. 2 confirms the assumed value of th diffusivity. Similar agreement for tests at ages of 7 days and 4 hours showed n significant change in the diffusivity within this range of age. In the test com menced at age 4 hours the measured temperatures exceeded the theoretical by maximum of about 4°C after 50 min owing to the liberation of heat of hydration When the internal temperatures were computed again with a diffusivity of 0·1 sq. in

min, but with an allowance for the internal generation of heat as described later, close agreement was obtained between the experimental and theoretical results.

The diffusivity obtained in these tests agrees with the value of 1 sq. ft/day which is often used for the computation of heat flow in massive dams. In those cases both the

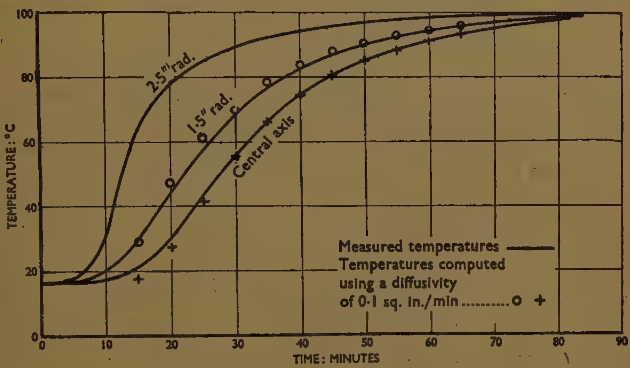

Fig. 2.—Two-dimensional flow of heat in a cylinder of concrete, of 6 in. dia., placed in steam at the age of 2 days

spatial and temporal dimensions are so great that it is convenient to quote the diffusivity in larger units.

Maturities at different points within the same specimen

Tests were undertaken to examine the temperature history of various points within the same specimen. Two, three, or four thermo-couples were cast in representative positions inside each of five specimens of differing sizes and shapes. Mature blocks were used so that the results would not be affected by the heat of hydration. The specimens were steamed for 200 min and then allowed to cool to room temperature. Varied temperature histories of the types shown in Fig. 1 were obtained, but the areas under the temperature/time curves (as shown cross-hatched in Fig. 1) were found to be practically equal for points within the same specimen. Although an inner point in a block of concrete experiences a slower and perhaps a smaller rise in temperature than a point at the surface, the conditions are reversed on cooling. At the end of the cycle, i.e., when the specimen is wholly at its original temperature, all points have gained the same maturity. This result can also be demonstrated from the theory of heat flow.

Maturities in specimens of different size and shape

Other tests were designed to examine the effects of size and shape on the gain of maturity obtained from a particular curing cycle. During a long treatment the internal temperatures will become constant even in quite large blocks and during these steady-state conditions both large and small specimens gain the same maturity. Interest thus centres on the conditions during heating and cooling, and the tests were therefore conducted in two stages.

Stage A.—Specimen, initially at 17°C, placed in steam for 200 min.

Stage B.—Specimen, initially at 100°C at all points, allowed to cool until a
points reached room temperature.

Fig. 3 shows the specimens subjected to these tests; where a third dimension is no
indicated it signifies that the test was staged in such a way that the flow of heat wa
two-dimensional. The results are shown in Table 1. Although certain trends ar
obvious they cannot be judged without the aid of a parameter to represent size an

TABLE 1

Specimen No.	Gain of maturity of central point: °C-hours		
	While heating Stage A	While cooling Stage B	Total
1	260	37	297
2	248	127	375
3	250	101	351
4	247	149	396
5	244	175	419
6	220	235	455
7	152	364	516
8	141	462	603
Infinitely small specimen	280	0	280

shape. In the case of a slab of finite thickness but infinite extent, initially at a uniform temperature throughout, if the exposed surfaces are instantaneously raise to and maintained at a higher temperature, the temperature/space/time relatio can be written[3] approximately in the form:

$$\frac{\theta_1-\theta}{\theta_1-\theta_0}=\Delta_a=\frac{\pi}{4}e^{-\frac{\pi^2}{4}\frac{h^2t}{a^2}}\sin\frac{\pi x}{2a} \quad \ldots\ldots \quad (1)$$

where θ_0 denotes initial uniform temperature
θ_1 ,, elevated boundary temperature
θ ,, internal temperature at distance x from boundary
h^2 ,, thermal diffusivity
t ,, time
and $2a$,, thickness of slab.

In this type of diffusion problem, a product solution[4] can be used so that for a rectangular parallelepiped:

$$\Delta_p=\Delta_a\times\Delta_b\times\Delta_c \quad \ldots\ldots \quad (2)$$

in which Δ_p is the proportional unaccomplished heating in the parallelepiped an Δ_a, Δ_b, and Δ_c are the corresponding quantities for infinite slabs of thickness $2a$, $2b$ and $2c$ respectively. Substituting equation (1) in equation (2) it is found that th time required for the central point of a parallelepiped having dimensions $2a$, $2b$, an $2c$ to reach a fixed proportion of the rise in surface temperatures is proportional t $\frac{a^2b^2c^2}{a^2b^2+b^2c^2+c^2a^2}$. Similarly, in the two-dimensional case, the time is proportiona to $\frac{a^2b^2}{a^2+b^2}$. These parameters are referred to here as shape–size factors or S–S

factors. The S–S factor for a cube is equal to 3(volume/surface area)^2 and for a long square bar (two-dimensional heat flow) it is equal to 2(sectional area/perimeter)^2. These expressions are used to calculate very approximately the S–S factors of shapes for which a formal approach would be impracticable. The S–S factors estimated in these ways are shown in Fig. 3.

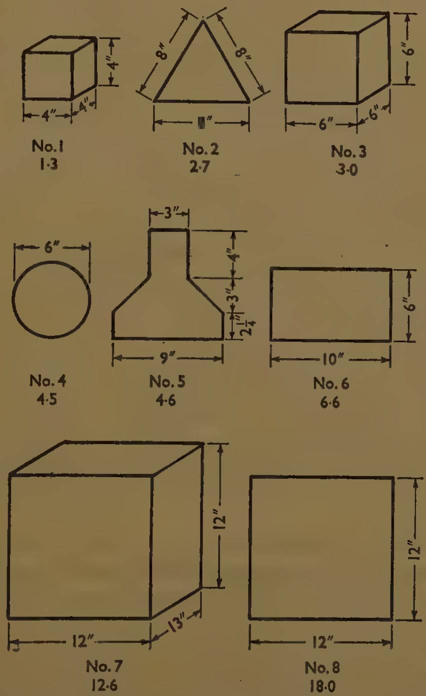

FIG. 3.—THE SPECIMENS AND THEIR S–S FACTORS

When the maturities quoted in Table 1 are plotted against the S–S factors, the curves shown in Fig. 4 are obtained. This diagram indicates well-defined relations between the gains in maturity during heating and cooling and the S–S factor. A word of explanation is necessary regarding zero S–S factor. This represents an infinitely small specimen which would *immediately* reach 100°C when placed in steam and *immediately* cool to room temperature on removal from the curing chamber. With a period of 200 min in steam and a room temperature of 17°C, this represents a gain in maturity of 280°C-hours, as shown in Table 1 and Fig. 4. The smaller gain of maturity during heating for increasing S–S factors and the larger gain during cooling are evident. Most significant is the fact that the total gain of maturity increases with the S–S factor. In practical steam curing the percentage increase will be much smaller than in Fig. 4 because these tests did not include a steady state which would add maturity equally to large and small specimens. Furthermore, the maturity is sometimes reckoned from a datum of -10°C and with this base the

numerical values for normal treatments are so large that the differences indicated in Fig. 4 become relatively unimportant.

It should be mentioned that the S–S factor cannot have a precise significance in this context since it represents to some scale the time required for the central point

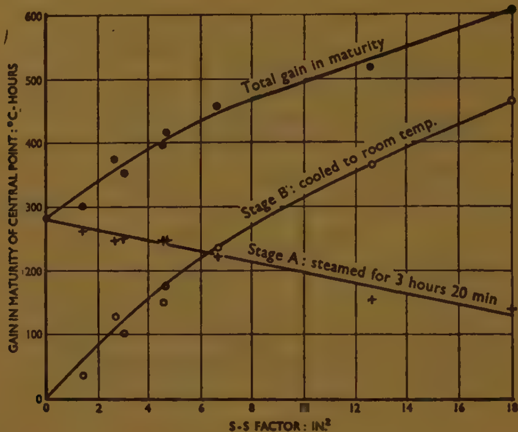

Fig. 4.—Relation between maturity and S–S factor for a particular curing cycle

of a solid to attain a certain proportion of an abrupt rise in boundary temperature. In practice, the boundary rise is not abrupt and the factor is used here to compare maturities and not times. Fig. 4 indicates, however, that it provides a good basis for such a comparison.

The range of S–S factors in these tests will cover most practical products and the important result emerges that the central point of a product will, at the end of a curing cycle, have achieved a maturity not less than that of cubes cured along with it. Since it is also known that all points within a block attain the same maturity, it is certain that in practice products will achieve the desired maturity.

Heat of hydration

The quantity of heat released when cement hydrates and the rate at which it is evolved are such that they assume importance only in massive structures. But the rate of evolution of heat depends on the temperature and thus, by steam curing, conditions are created in which most of the heat of hydration might be released in a short time and an appreciable additional rise in temperature could result.

A limited investigation of this matter was made by studying the evolution by the heat-of-solution method while the cement was cured at 18°C, at 50°C, and at 100°C. The shape of the heat/time curves showed the usual exponential form rising to a value of 65 cal/gm after 330 hours at 18°C. The rate of evolution, however, depended greatly on the curing temperature; at 18°C 30 cal/gm had been released after 36 hours, whilst at 100°C the same quantity of heat had been evolved after only 2 hours.

The effect of this acceleration in the evolution of heat was calculated in the case

of a long 12-in. × 12-in. block of 3 : $1\frac{1}{2}$: 1 concrete, of water/cement ratio 0·45, by an iterative numerical procedure referred to in the Appendix. It was assumed that the boundary was abruptly raised to and maintained at 100°C and, for each time interval in the computation, the rise in temperature due to the liberation of heat of hydration at the appropriate rate was estimated from the quantity of cement in each unit volume, and from the specific heat of the concrete.

This computation showed that the release of chemical heat accelerated the achievement of a particular temperature at the centre. For example, 50°C was reached 15 min earlier and 80°C 50 min earlier than in the same case without internal generation of heat. The additional maturity, however, due to this effect was not significant compared with the total. It was also found that the computed internal temperature rose to a maximum of about 105°C and then declined. The calculation seemed to show that if the release of chemical heat were able to raise the internal temperature above that of the boundary, the excess would probably never be great because of the relative ease with which heat can flow out in reverse to the boundary in specimens of the usually moderate size which are steam cured. The large S–S factor, rich mix, and abrupt rise in boundary temperature assumed in this hypothetical case are severe and not likely to arise in practice, but temperatures up to 103°C were recorded experimentally in some specimens which were subjected to drastic thermal treatment immediately after casting.

Under practical conditions, however, there is usually a waiting period before treatment and, with the lower temperatures and gradients employed, the evolution of internal heat will be more gradual than in these tests. It seems probable that the only effect of the heat of hydration will be a small increase in the maximum internal temperature above that of the boundary and a modest gain in maturity above that created by steaming alone.

Conclusions

In a complete cycle of heat curing, i.e., a period sufficient to allow cooling uniformly to the original temperature, the gain in maturity of all points inside a concrete block is the same.

In a complete cycle, large blocks achieve a maturity somewhat greater than small ones. Therefore small cubes have no unfair advantage over the products, the strength of which they are intended to represent.

The heat of hydration of ordinary Portland cement is unlikely to increase significantly the maximum temperature during steam curing under practical conditions. It accelerates the rise in temperature but the modest increase in maturity which results is not important compared with the total maturity obtained from a curing cycle.

Acknowledgement

The Author's thanks are due to Mr M. G. Handcock, M.Sc.(Eng.), for his valued contribution to this investigation.

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APPENDIX

The integral of time and temperature only has been considered. The temperature/time gradient, however, affects the strength of concrete and the temperature/space gradient may also be influential. These may be determined experimentally by using numerous thermo-couples, but it is obviously desirable to be able to predict the internal temperature space/time relation theoretically. This can be done provided the diffusivity and the variation of boundary temperature are known. The general equation for the conduction of heat in solids has been solved for certain simple geometrical shapes and variations of boundary temperature. Irregular shapes, however, with practical variations of boundary temperature are not amenable to formal mathematical treatment, and for these it is necessary to use approximate methods which are usually either analogical or numerical. For the present work an iterative numerical procedure[5] based on Schmidt's[6] method has been used. The area or volume is divided by a mesh of equidistant stations and the appropriate variation of temperature at the boundary is fed into a step-by-step calculation of the temperatures throughout the solid. It is desirable to arrange that the factor $h^2\Delta t/(\Delta x)^2$ has the value of $\frac{1}{2}$, $\frac{1}{4}$, or $\frac{1}{6}$ in one-, two-, or three-dimensional cases respectively. Here h^2 is the diffusivity, Δt a time increment, and Δx a space increment between stations, and by a suitable choice of increments it is easy to ensure that the factor has the appropriate value. Provided this condition is observed, the computation reduces to the extremely simple process of averaging the temperatures at adjacent stations, in order to find the temperature at a particular station after the lapse of a time interval Δt. The choice of these particular values of $h^2\Delta t/(\Delta x)^2$ not only simplifies the numerical procedure, but it yields a stable solution which does not diverge or oscillate. The method is flexible since it can be used with any shape, with any variation of boundary temperature and with the internal generation of heat.

This type of solution lends itself well to graphical procedures in the one- and two-dimensional cases. Since the process is essentially one of averaging, the intersection of straight lines through vectors representing the temperature can replace numerical addition and division. Even the graphical construction can be replaced by a simple labour-saving mechanical procedure in which a cross-section of the heated body with its array of stations is drawn on paper, pinned to the drawing board. Flag pins, which initially mark the stations, are moved in steps out along diagonal vectors which represent the temperatures at the stations. Fine threads of dark elastic, stretched between the flag pins, serve as convenient extensible straight edges, by means of which the vectorial temperatures are averaged. In this manner, problems in the transient flow of heat can be solved rapidly and without fatigue, and with an accuracy sufficient for most engineering purposes. Fig. 1 was obtained in this way.

The Paper, which was received on 20 June, 1956, is accompanied by four sheets of diagrams, from which the Figures in the text have been prepared, and by an Appendix.

Correspondence on this Paper should be forwarded to reach the Institution before 15 February, 1957. Contributions should not exceed 1,200 words.—Sec.

Paper No. 6143

SOME CIVIL ENGINEERING ASPECTS OF ATOMIC POWER GENERATION

by

*** Ian Davidson, M.Eng., M.I.C.E.**

(Ordered by the Council to be published with written discussion)

SYNOPSIS

The Paper gives a brief outline of the theory of reactor shielding and a short description of the principal features of the graphite-moderated gas-cooled reactors now under construction by the United Kingdom Atomic Energy Authority. The general arrangement and economics lead on to the structural design of the biological shield and factors on which it is based. Further developments are indicated, and the roof slab is described.

The loads to be carried by the foundations and the settlement specification for the United Kingdom Atomic Energy Authority reactors are outlined, with a brief description of the foundation arrangements.

The steel tower for the construction of the biological shield is mentioned, and cranage and other site facilities for plant fabrication and erection are described. The effect of erection facilities on site layout is described and illustrated by reference to the layouts of the two United Kingdom Atomic Energy Authority atomic power stations.

Introduction

The United Kingdom Atomic Energy Authority are now building two nuclear power stations, each of four reactors. The White Paper Plan starts with the construction in 1957 of two further stations, each of two reactors, by the Central Electricity Authority. These are to be followed by two similar stations 18 months later. These sixteen reactors will be of the graphite-moderated gas-cooled type, and this Paper will therefore be entirely concerned with this simple and safe type of reactor.

The reactors now being built by the United Kingdom Atomic Energy Authority constitute the first substantial nuclear power plants ever constructed; but the reactors to be built by the Central Electricity Authority will be of improved types, in which the civil engineering problems may well be modified.

The basic principles and a brief description of the graphite-moderated gas-cooled reactors now being built at Calder Hall were included in the 1955 James Forrest Lecture.[1] In order that the problem of shielding can be understood, the following brief notes on one aspect of nuclear physics must first be considered.

Nuclear bombardment

The fuel elements in the core of these reactors, when working, are bombarded by great numbers of neutrons, and as a result of the fission of the uranium fuel fresh

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[1] Sir John Cockcroft, "The development of nuclear power" (James Forrest Lecture 1955). Proc. Instn Civ. Engrs, Part I, vol. 4, p. 791 (Nov. 1955).

neutrons are produced. These are fast neutrons, but in passing through th graphite moderator their velocities are reduced to correspond with the temperatur of the moderator. They thus become slow, or thermal, neutrons.

Thermal neutrons are fairly readily captured by atoms of some of the light element and this capture generally results in the emission of a gamma ray. This is a form of energy, and may be visualized as a very powerful X-ray. Gamma rays are mos easily stopped by heavy matter. The intense radiation of fast and thermal neutron and gamma rays from the reactor would be very dangerous to operatives and shielding is required.

Shielding

The best arrangement of the shielding is not easily determined since gamma need heavy matter, such as steel, whilst the neutrons require light elements, such as the hydrogen in the chemically combined water in the concrete. Moreover neutrons in the shield will tend to be changed to gamma rays, and heat is produced both by the change from fast neutron to thermal neutron and also by the decay o the gamma ray. Owing to the poor thermal conductivity of concrete it is essentia to keep down the amount of heat generated in it, or dangerously high temperature and thermal gradients may occur.

The compromise which is used in the Atomic Energy Authority's reactors now under construction is to use an inner shield of 6-in. steel plates; the bulk of the hea is produced in the steel, whence it is readily removed by air cooling of the fron and back faces. This is called the thermal shield. It is surrounded by 7 ft of concrete, in which the neutrons and the secondary gammas are trapped, so that the intensity of radiation at the external face is brought down to a level which is biologically safe. This is the biological shield. Since it has to stop both types of neutron and secondary gamma rays, it relies both upon its combined water and upon as high a density as can conveniently be attained.

The graphite-moderated gas-cooled reactor

The general arrangement of the reactors being built by the United Kingdom Atomic Energy Authority at Calder Hall in Cumberland, and at Chapelcross near Annan, may be seen from Fig. 1. In the centre is the graphite moderator weighing more than 1,000 tons, which is contained in a cylindrical pressure vessel about 40 ft in diameter and approximately 60 ft high. The pressure vessel is supported on ten inverted A-frames, which rest on grillages bolted to the main foundation. Emerging from the top dome of the vessel are the charging tubes, through which the fuel elements are lowered into and removed from the reactor.

Surrounding the pressure vessel is the concrete biological shield, the inner face of which is protected by the thermal shield. Induced-draught air cooling is supplied to the spaces between the thermal shield and biological shield, on all sides, and the outlet air is exhausted 200 ft above ground level from the two stacks on the pile building.

It will be further seen from Fig. 1 that the hot gas from the top of the pressure vessel is carried in four steel ducts to the tops of the four cylindrical heat exchangers, which stand on end around the reactor. These heat exchangers each consist of a steel pressure shell, about 18 ft in diameter and about 70 ft high, containing the tubes through which water is circulated to produce superheated steam. The cooled gas is drawn from the bottom of each heat-exchanger shell and led to a centrifugal

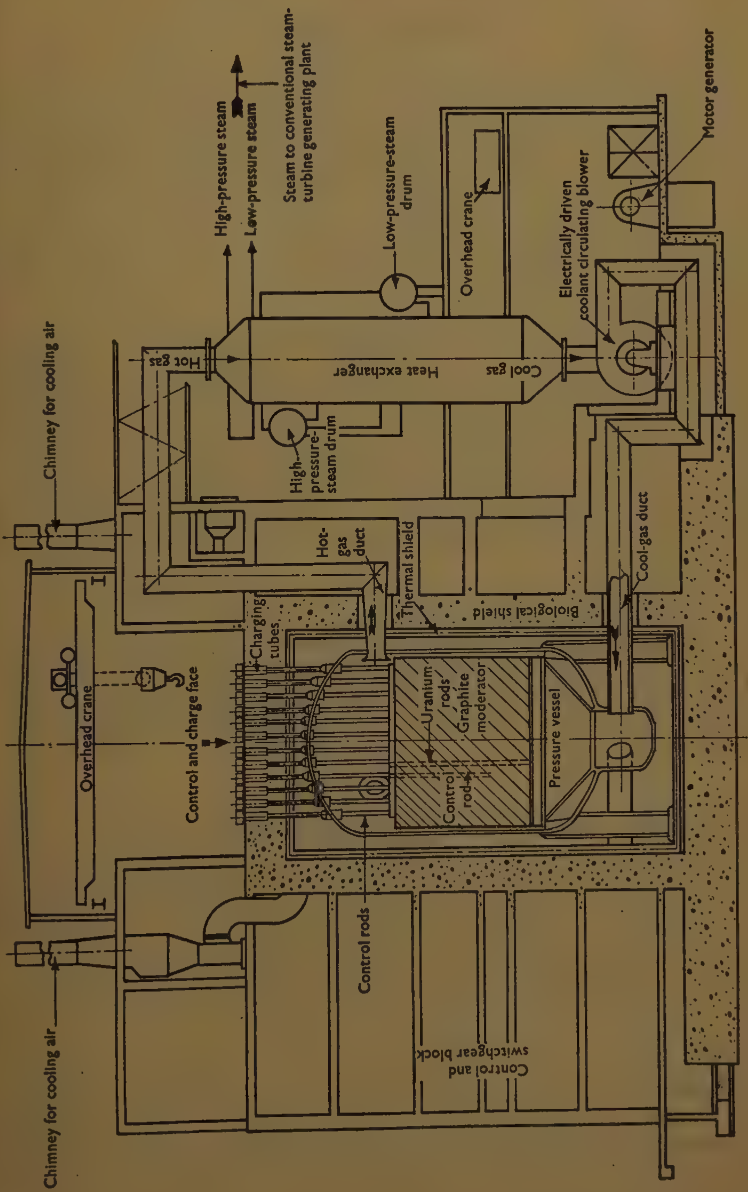

FIG. 1.—GENERAL ARRANGEMENT OF REACTORS AT CALDER HALL AND CHAPELCROSS

blower of 2,000 h.p. which returns it to the inlet manifold at the bottom of the pressure vessel.

The biological shield

General arrangement

The basic dimensional requirements for the biological shield now become clear. The main concrete structure is octagonal in plan, 46 ft across from side to side internally and about 85 ft high, with a thickness of 7 ft. This stands on a concrete slab which must also carry the load of the pressure vessel and contents, about 1,600 tons, distributed over ten grillages. There is a similar 8-ft-thick concrete roof slab across the top, resting on the octagonal walls and spanning between them and pierced with numerous holes through which the charge tubes pass. Furthermore, thinner subsidiary biological-shield walls are required around those parts of the gas ducts adjacent to the pressure vessel.

The choice of an octagonal plan for the main shield gives relative freedom for thermal movements and convenience of shuttering, together with a practicable shape for the steel thermal-shield plates which line it. The subsidiary biological-shield walls are, however, designed to fulfil some other useful purposes besides their basic one. In the case of the reactors at Calder Hall the nature of the ground makes it necessary to spread the load over a large raft, measuring 130 ft × 104 ft, and the design of this raft is obviously difficult since the bulk of the load is concentrated around an octagonal figure at its centre. The subsidiary biological-shield walls, or wing walls, are therefore so arranged that they act as stiffening brackets to the upper surface of the raft.

Furthermore, these wing walls serve to enclose parts of the blower houses and to provide machinery platforms at various heights, culminating in the plinths for the self-supporting steel ventilating shafts. These plinths are 110 ft above ground level. In short, it can be said that the concrete octagon and the concrete box construction surrounding it serve as a structural spine, supporting not only plant and equipment but also the five floors of the structural steel-framed control building and discharge building, which abut upon it.

Economics

The structural usefulness of the biological shield has been mentioned above because it is a factor which must be taken into account when considering the economics of using a concrete shield wall. Comment has been made that the concrete is very costly, but it must be remembered that the building and civil engineering construction in a nuclear power plant forms only a rather small proportion of the whole cost. The biological shields, complete with footings, which are being built at Chapelcross to the specification outlined above cost less than 5% of the total project cost.

Structural design

As already mentioned the shield concrete should have a substantial chemically combined water content, and also have a high density. In order to meet these requirements the Atomic Energy Authority's specifications call for a mix of approximately 4 : 2 : 1 (3,000 lb/sq. in. at 28 days) and an oven-dry density of at least 150 lb/cu. ft. This latter has been met by the use of a quartz dolerite (whinstone) coarse aggregate which has a high specific gravity. Internal vibration is also used since it produces a definite increase in concrete weight. It is apparent that cracks

in the concrete, if they afford a straight path through the shield, will render it less satisfactory in use. Precautions are therefore taken against shrinkage by bringing the concrete up in separate blocks on a predetermined plan, and both horizontal and vertical construction joints are joggled. In order to reduce shrinkage a dry mix has been used, with a plasticizer to aid consolidation.

The structural design of the concrete biological shield must take into account the known loads which come upon it, and also the effect of temperature. The heat radiated from the pressure vessel and gas ducts may be ignored, since they are well lagged and cooling air circulates over them. But, as mentioned in the introduction, heat is produced in the concrete itself by the radiation which passes the thermal shield and strikes the concrete.

The biological-shield walls were originally designed on the basis of a straight-line temperature curve, with the inner face 20°F hotter than the outer. The temperature conditions in the shield roof were rather higher. The only design difficulties appeared where restraint prevented the free movement of the concrete, principally at the junction between the walls and the floor and roof slab, since no expansion joint could be permitted.

Later forecasts of concrete temperatures somewhat exceeded those quoted above, and the point of maximum temperature was theoretically determined to be near to, but not at, the inner face, which is air-cooled. The concrete temperature should rise from the inner face at a maximum gradient of about 40°F/ft, then falling off quite gradually to the outer face. The effect of such a localized temperature curve is not easy to forecast since such factors as creep, thermal expansion, and thermal conductivity in concrete are uncertain. However, it is considered that the theoretical tensile stress in the concrete will not exceed a safe value, and of course the inner face of the shield is well reinforced in both directions.

The steel thermal shield is considerably more expensive than an equivalent thickness of concrete, and it would be a real advantage if a concrete shield could be developed which could stand alone. This possibility will depend upon the ability of concrete to stand up to the full radiation current, and upon it being able to withstand the temperatures.

Roof slab

The roof slab which spans the pressure vessel forms one of the most intricate items of the civil engineering work, both in design and construction. It is perforated with a very large number of holes, in which are the charge tubes. When the pressure vessel is hot the charge tubes will move, and must be given complete freedom to do so. The clearance between the charge tube and shutter-tube is therefore eccentric when cold, and no two pairs are alike.

The space left for the main reinforcing bars is very restricted, but a satisfactory design has been produced in which the 8 ft thickness is poured in four lifts, the slab being self-supporting when the third lift has hardened. The method of construction is similar to that used in the Windscale piles.[2] Temporary Bailey Bridge girders span over the octagon and support hanger rods. To the bottom of the rods is fixed a framework of steel sections in which the thermal-shield plates are supported. Permanent steel shutter plates are then hung from the rods and adjusted to the right camber; the shutter-tubes for the charge tubes are assembled and reinforcing bars

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[2] D. R. R. Dick, "The civil engineer and Britain's atomic factories". Proc. Instn Civ. Engrs, Part III, vol. 4, p. 514 (Aug. 1955). See p. 523.

threaded through. Concreting is then carried on until after the completion of t third lift, when the hanger rods may be cut off and the Bailey Bridges dismantlec

THE FOUNDATIONS

The total load imposed on its foundation by the concrete thermal shield describ above amounts to 24,000 tons, including plant and equipment loads. There is al a stringent settlement condition to be met, which arises from the fact that the pre sure vessel is connected by gas ducts to the heat exchangers and to the blowers, whil these are too far removed from the pressure vessel to be on a common foundatio The flexibility of the ducts is only sufficient to allow $\frac{1}{8}$-in. differential settlement b tween heat exchanger and blower, and $\frac{1}{4}$-in. differential settlement between pressu vessel and blower or heat exchanger. The closing pieces of these gas ducts a fitted about 12 months after the main loads come on the reactor raft.

It is clear that with such a wide range of loads a very cautious policy is requir to guarantee that the differential settlements will be so small. At Calder Hall t foundations are in a glacial moraine having a low silt content but including so clay lenses, with sandstone rock about 60 ft down. It was decided to keep t ground pressure down to $2\frac{3}{4}$ tons/sq. ft at a depth of 16 ft below ground level an that required a raft 130 ft × 104 ft, as has already been mentioned, under the react itself.

Although this raft is stiffened by bonding the subsidiary shield-wall system into as previously described, it was necessary to make it 11 ft thick, and the adequa reinforcement of such a section with round steel bars is quite an undertaking. It i interesting to note that this raft weighs 40% of the load which it carries.

The foundation problem at Chapelcross is very much simplified since sandsto rock underlies the site at a comparatively shallow depth and the main loads can l carried directly down to it.

ERECTION FACILITIES

Biological shield

The inner face of the concrete biological shield is required to be finished with $\frac{1}{8}$-in. tolerance of the line shown on the drawings. There are also a number sleeves through which plant items pass, together with brackets for the thermal-shie supports, and other fixtures which must be accurately located. In order to ensu a stable platform, from which the octagon corner shutters were strutted, and c which the survey lines and levels could be carried up, a special structural steel tow was provided at Calder Hall which occupied the entire interior of the octago Satisfactory results are being obtained at Chapelcross with climbing shutters alon

Pressure vessel

The erection of some of the plant items demands interesting cranage and co struction facilities. The plates for the pressure vessel are assembled and but welded at site, to form the bottom and top domes and two ring sections, togeth with the diagrid. In order to permit the best conditions for this work, which carried out to Lloyds' Class 1 fusion-welding standards, plinths are provided on whic each item can be fabricated. The two domes are enclosed in portable she 47 ft × 47 ft × 27 ft high, with domed roofs which can be lifted off by a crane permit the plates to be lowered in.

On completion of the bottom dome, which weighs nearly 100 tons, it is jacked c

its plinth, on which it has been fabricated in an inverted position, and lowered on to a trolley running on a 26-ft-gauge track. This brings the dome under a 100-ton guy derrick, having a 125-ft mast and 100-ft jib mounted on an 80-ft steel tower; this is erected near the biological shield, which has meantime been completed to its finished height. The bottom dome is then lifted by the derrick, inverted to its correct attitude, raised over the 110-ft-high concrete shield, and lowered on to the A-frame legs which have previously been placed in position.

The two ring sections, the diagrid, and the top dome follow similarly, and are all welded together to complete the pressure vessel. The stress relieving of the completed vessel is achieved by radiant heating, which requires 1·5 mW of electricity. At Chapelcross it is intended to mount the guy derrick actually on the top of the biological shield.

The heat exchangers

The heat exchangers also require considerable site facilities since they are delivered in the form of rings 18 ft in diameter, six of which, together with two dome ends, form a complete shell. The site welding of these vessels is also done to Lloyds's Class 1 fusion-welding standards and the rings are lifted on to bogies running on a long track; the rings rest on rollers so that they can be rotated and the bogies are run into a temporary shed 45 ft × 100 ft × 25 ft high, which accommodates two lines of track. When the circumferential welds have been completed they are stress-relieved individually by high-frequency cables wrapped around each weld. The completed vessel is then run out of the far end of the shed and jacked on to a steel grillage, where it is tested by hydraulic pressure.

The shells, which weigh about 200 tons, are then taken by low-loader to the required position for erection. This is done by two gin-poles which lift the top end of the vessel while the rear end slides in on timber baulks; in the last stages the vessel is lifted vertically on to its plinth. At Chapelcross the heat exchangers will be fabricated inside an existing aircraft hangar 100 ft × 180 ft, the roof of which will have to be raised 9 ft; this will take four assembly lines.

Derrick cranes

In order to handle the various materials for the biological shield the civil engineering contractor needs cranes on the control and discharge faces of the reactor. These will also be required by the structural steelwork contractor for the control and discharge buildings. The erection of the upper carbon-dioxide-gas ducting will provide further use for these cranes, and in fact it is the lifting of the top isolating valves in these ducts that finally determines the requirements. This has led at Calder Hall to the provision of one 7-ton Scotch derrick with 120-ft jib on tracks, at each face, later augmented by one 10-ton Scotch derrick with 120-ft jib on 44-ft gabbards, on tracks, at each face.

SITE LAYOUT

The principles normally applicable to the layout of a steam generating station will apply also in the case of a nuclear power plant, but some of the constructional features just described should also be taken into account. The site layouts of Calder Hall and Chapelcross are shown in Figs 2 and 3; these will serve to illustrate these considerations and to show how experience, and the freedom of a larger site, has made the Chapelcross layout rather more convenient than that at Calder Hall.

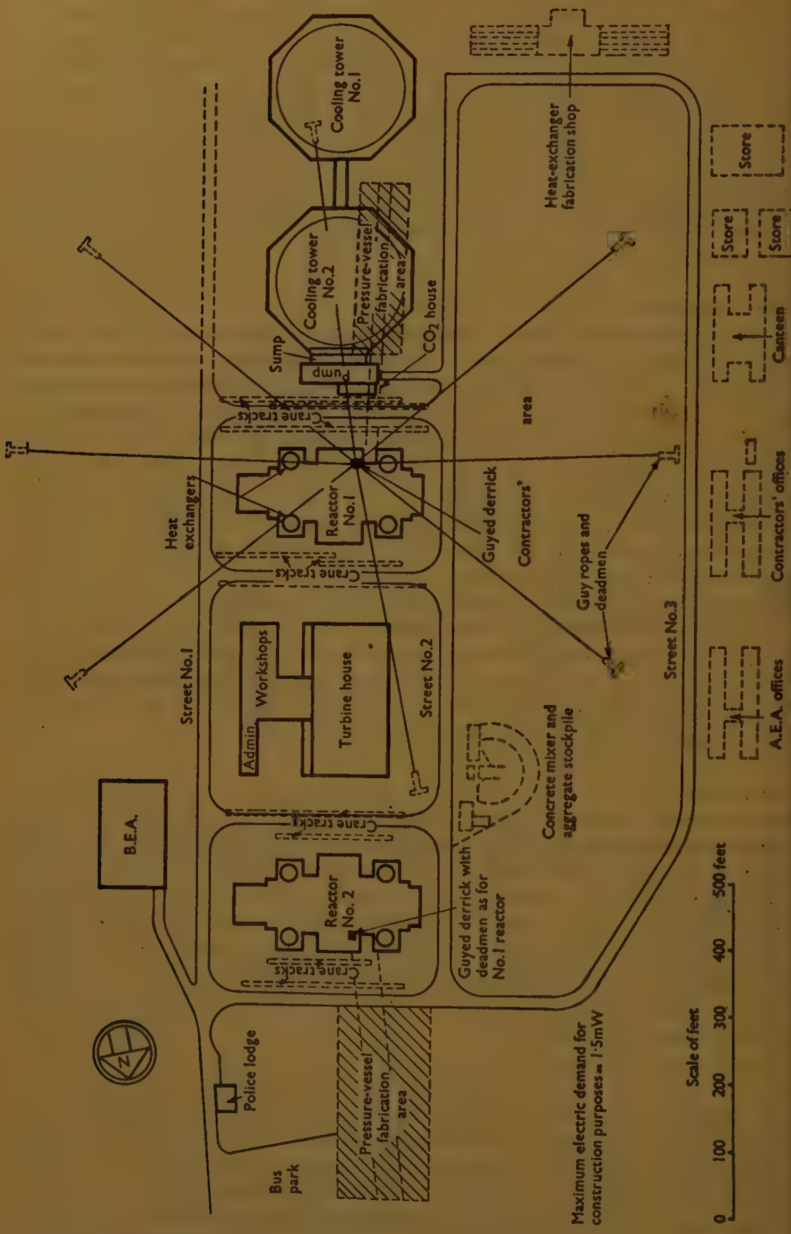

FIG. 2.—SITE LAYOUT. CALDER WORKS

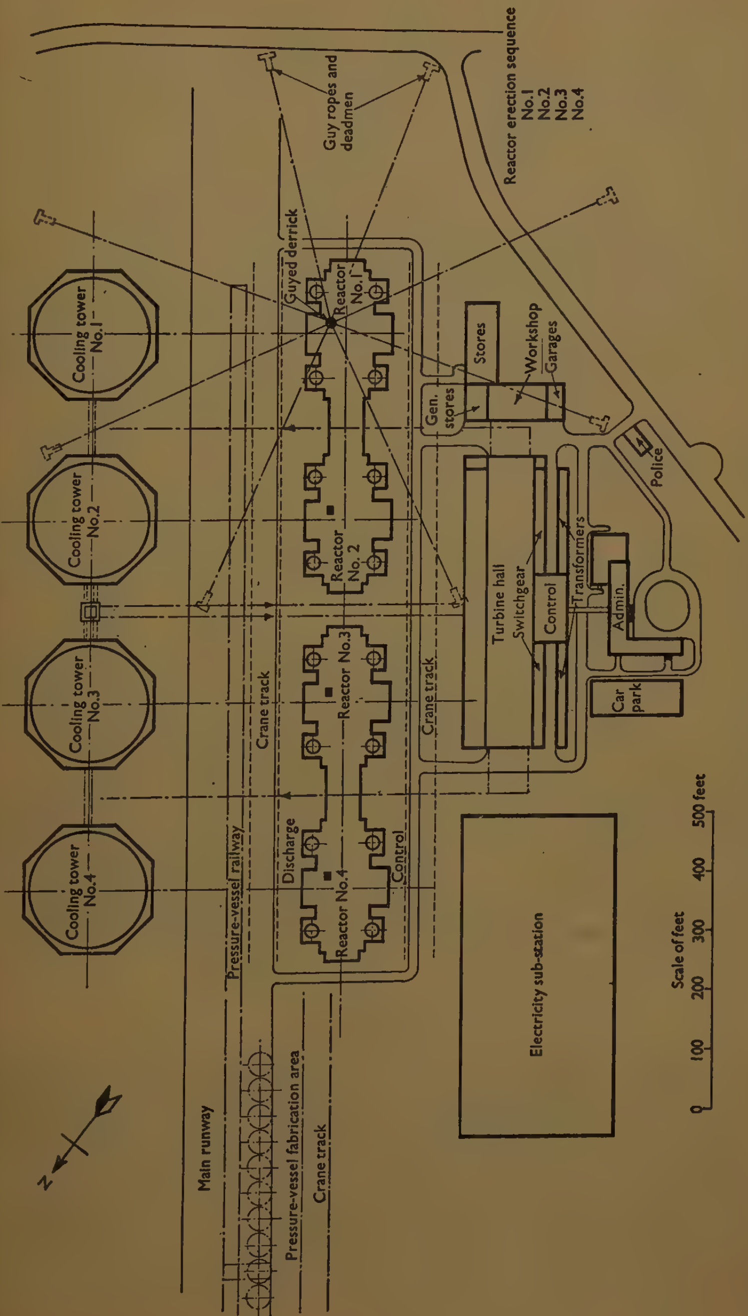

FIG. 3.—SITE LAYOUT. CHAPELCROSS WORKS

The site at Calder Hall was originally laid out to accommodate two reactors, o which only one was authorized in the first place. The particular grouping of the mai structures suited the shape of the site and permitted the construction sequence t work from south to north.

Having regard to its effective radius the only possible position for the 100-to derrick is at the discharge face of the reactor. The guy ropes for this derrick ar about 600 ft long and there are eight of them; these have decisive importance a regards the siting of other structures and services, which must be carefully con sidered both in space and time. There are also guys for the gin-poles to lift th heat exchangers, and space must be left for the heat-exchanger shells to brough into their correct positions for lifting.

The derrick cranes which run on the control and discharge faces of the reacto occupy valuable space, which is not released until comparatively late in the pro gramme because they are required to lift the top gas ducts. The area required fo the pressure-vessel fabrication is quite considerable and at Calder Hall it had to b quite close to each reactor, so that the sections could be run under the guy derrick The fact that the heat-exchanger shells are brought into position on a low-loade gives complete freedom to the siting of their fabrication shed; the only requiremen is a sufficient length of level track, with good road access.

The further extension of the Calder Hall site, to accommodate two more reactors is towards the north, where the shape of the site makes the layout similar to tha already developed. Chapelcross is on a former R.A.F. airfield and only a portion o this is required for the atomic power station, so that there is much more freedom i laying out the site.

As will be seen from Fig. 3 the four reactors at Chapelcross are lined along the edg of the former main runway, towards which all their discharge faces are turned. Ther is a common fabrication area for all the pressure-vessel sections and the latter will b traversed on to a permanent 26-ft-gauge track laid down the runway. From this the will be traversed back under the derrick crane for lifting into position.

This scheme provides an advantage in the vessel fabrication, which is brought int one place, with plinths which will be used repeatedly, and where supervision and provision of services will be more convenient. Also the track on which the vesse sections move is quite clear of the tracks on which the derrick cranes operate, so that neither hinders the other. It will also be noted that the derrick-crane track can be extended from reactor to reactor, so that the cranes can be traversed from one to another, without the expense and time of dismantling and re-erecting.

Conclusion

It has only been possible to survey rather briefly in this Paper some of the civi engineering considerations peculiar to the graphite-moderated gas-cooled reactor now under construction by the United Kingdom Atomic Energy Authority. There has been, of course, some other interesting civil engineering work at these sites, bu no attempt has been made here to touch upon features which are common to any power station. It is also clear that improved designs and types of reactor will pose further civil engineering problems in the future, and these may well require a more accurate knowledge of the effect on, and the behaviour of, materials in atomic reactors.

ACKNOWLEDGEMENT

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The Paper, which was received on 29 June, 1956, is accompanied by one sheet of drawings from which the Figures in the text have been prepared.

CORRESPONDENCE on this Paper should be forwarded to reach the Institution before 15 February, 1957. Contributions should not exceed 1,200 words.—SEC.

Paper No. 6154

THE USE OF FLY ASH AND SIMILAR MATERIALS IN CONCRETE

by

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and

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(*Ordered by the Council to be published with written discussion*)

Introduction

Fly ash is the fine ash obtained by electrical or mechanical precipitation when pulverized coal is burned in a steam boiler plant. The essentials for a good fly ash are consistency in chemical analysis, unburnt residue, and fineness. Absolute consistency is difficult in a material which even when obtained from the same source is probably produced from a fuel which comes from different sources, from boilers which work at varying rates, and from a precipitating plant which may not have been functioning consistently.

Before the North of Scotland Hydro-Electric Board decided to use fly ash as a partial substitute for cement in the concrete of such an important structure as a dam an investigation, including a visit to large hydraulic structures in the United States where it had been used, was made by the Authors.

Apart from the advantage of saving cement, one of the advantages which had been claimed for the introduction of fly ash into concrete mixes was that it produced in the resulting concrete less heating during setting, smaller shrinkage, and less vulnerability to acid waters than ordinary cement. In Scotland, where dams are required to retain aggressive waters, it has long been the practice to make the impermeability of the concrete the criterion rather than its strength. This had generally been achieved by maintaining a high proportion of cement in the concrete used for the facing of a dam. To reduce the amount of cement would only be acceptable if it were replaced with some substance with qualities such as are claimed for fly ash.

In the United States, on the other hand, the introduction of fly ash into concrete was not brought about by any apparent scarcity of cement, nor as a convenient way of disposing of an unwanted waste product, but because of the strong trend in America towards lean mixes. This tendency had become somewhat inevitable by the general practice to specify a concrete mix on a strength basis. It also seems to have come about as an anti-heat measure. Great trouble is taken in the United States to reduce the heating during setting if, thereby, concrete can be placed faster. It may have been partly this characteristic of fly ash which tempted them to use it in some substantial dam structures. The factor which has prevented mixes from becoming leaner still has been the need to maintain reasonable workability.

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Apart from reducing heating during setting, fly ash also possessed the advantage of ensuring no loss of workability when it was used to replace cement.

The background for the introduction of fly ash in the United States and in Scotland is therefore quite different and, since impermeability has been of so much importance there, the mixes against which any substitution must be judged are radically different in the two countries. For example, in the United States, the Corps of Engineers for the dam structures for which they were responsible started off with a cement requirement for hearting concrete of about 282 lb/cu. yd but later, to control the temperature rise, reduced this to, in one case, 212 lb. and in another, 188 lb.; for face concrete 376 to 423 lb/cu. yd was required. The Bureau of Reclamation in such structures as the Hoover, Grand Coulee, and Shasta dams which were built before the 1939–45 war used 376 lb/cu. yd and on reservoir 22 (Denver) 282 lb/cu. yd. These figures compare with from 336 to 448 lb/cu. yd for hearting concrete and 560 to 670 lb/cu. yd for face concrete in the United Kingdom. In terms of workability, these figures are not entirely comparable for, in the United States, it is the invariable practice now to use an air-entraining agent in all their concrete. This gives the Americans a higher degree of workability than is possible in the United Kingdom where air entrainment is not used.

Fly-ash concrete is the concrete produced when a quantity of the cement normally used in a mix is replaced by an equal amount by weight of fly ash. It does not refer to concrete in which fly ash is used as a replacement of some of the sand in the mix without any adjustment in the quantity of cement. In concrete, fly ash behaves as a pozzolana. It has a high silica content, and combines with the lime set free when concrete sets. This action takes time and is the reason why fly-ash concrete is not suitable in cases where it is important for concrete to reach its maximum strength quickly.

Much research has been done over a long period into the use of fly ash, as is shown by the large number of publications[1] which have already appeared on the subject. Most of the laboratory work into the use of fly-ash concrete in hydraulic structures has been carried out by the following at the places named:—

University of California	by Professor Davis
U.S. Bureau of Reclamation	at Denver (Colorado)
U.S. Corps of Engineers	at their laboratory at Jacksonville
American Portland Cement Association. .	at Chicago
Ontario Hydro-Electric Commission. . .	at their laboratories in Toronto
University of Glasgow.	by Professor Marshall

The general conclusion of all these investigations was that, provided the fly ash had a low enough carbon content, and a high specific surface, it was a suitable ingredient to use in replacement of cement in the concrete of dams. At the University of California, Professor Davis's view, based on his researches, was that a carbon content of 7% with a permissible deviation of 1% was admissible. He emphasized the importance of fineness and, from tests of a number of European (not British) fly ashes, had found them too coarse for use as pozzolanas. He had studied permeability particularly and was satisfied that the permeability factor for fly-ash concrete improved very considerably with age; e.g., at 28 days fly-ash concrete was three times as permeable as normal concrete, but after 6 months it was less than one-quarter as permeable. This characteristic is important for dams, where the concrete is seldom

[1] "Fly ash as an admixture in cement and concrete" (Bibliography), Cement & Concrete Assocn, publ. Ch. 27, Dec. 1952.

TABLE 1

(Summarized from Papers by Raymond E. Davis *et al.*, in Journal American Concrete Institute, May 1937 and January 1941)

(a) General investigation of different ashes

Cement: Pozzolana: %	Cement: Pozzolana	Carbon in fly ash: %	Neat paste, Time of setting (Gillmore): Initial: hr. min.	Neat paste, Time of setting (Gillmore): Final: hr. min.	w/c ratio by weight	Compressive strength of concrete*: lb/sq. in. (3-in. × 6-in. cylinders): 7 days	28 days	3 months	1 year
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
0	None	—	2 15	4 00	0·43	4,040	5,310	5,790	6,810
10	Chicago fly ash	1·1	2 45	4 40	0·41	4,010	5,260	6,110	7,500
20	Chicago fly ash	1·1	3 00	5 00	0·40	3,760	5,190	6,230	7,610
	Cleveland fly ash	1·1	3 40	5 00	0·42	3,720	4,510	5,700	6,760
	Indiana fly ash	3·3	3 10	5 00	0·41	3,540	5,010	6,330	7,190
	West Penn. fly ash	6·4	3 05	5 10	0·43	3,370	4,910	6,260	7,450
	Union Elec. fly ash	7·4	3 40	5 45	0·45	3,330	4,510	5,620	6,410
	Detroit fly ash	7·5	3 25	5 35	0·44	3,170	4,790	5,870	6,690
	Duquesne fly ash	9·3	3 30	4 45	0·48	2,890	4,220	5,200	6,130
	Long Island fly ash	10·4	4 10	6 30	0·46	3,220	4,910	6,390	7,080
	Potomac fly ash	11·9	3 25	5 20	0·45	3,270	4,770	5,830	6,840
	N.Y. Edison fly ash	13·6	3 25	5 25	0·49	2,940	4,310	5,360	6,240
	Cos Cob fly ash	13·9	3 45	6 00	0·49	2,830	3,770	5,400	6,540
	Stamford fly ash	15·7	4 55	6 45	0·47	2,980	4,100	5,840	6,550
	N.Y. Steam fly ash	16·6	3 45	5 50	0·49	2,820	4,350	5,330	5,930
	Hell Gate fly ash	19·3	3 00	5 25	0·49	2,810	3,900	4,940	5,720
	Boston fly ash 1	26·4	3 30	5 20	0·50	2,620	3,950	4,970	5,840
	,, ,, 2	30·2	3 50	5 45	0·52	2,640	3,830	4,700	5,330
	Average for sixteen fly ashes	12·1	3 35	5 30	0·46	3,090	4,440	5,600	6,520

TABLE 1—*continued*

30	Chicago fly ash	1·1	3	20	6	20	0·40	3,210	4,970	6,400	7,770
	West Penn. fly ash	6·4	3	30	6	15	0·43	2,850	4,400	5,900	7,390
	Long Island fly ash	10·4	4	25	6	45	0·48	2,640	4,250	5,800	6,730
	Potomac fly ash	11·9	3	35	6	05	0·46	2,590	3,960	5,250	6,360
	Limestone dust	—	2	00	3	25	0·43	3,120	4,030	4,680	5,320

* Cement/aggregate ratio = 1 : 5·6 by weight; 0 to $\frac{3}{4}$-in. aggregate; flow 60%.

(b) Effect of varying percentage of fly ash on strength

	Fly ash			Fly ash: %	Ratio of compressive strength of concrete containing fly-ash cement to compressive strength of concrete containing corresponding Portland cement: %						
					General purpose cement				High early strength		
	Type	Carbon: %	Specific surface: sq. cm/gm		7 days	28 days	3 months	1 year	7 days	28 days	3 months
Fly-ash cement	Chicago . . .	1	3,220	10	99	99	106	110	—	—	—
				20	93	98	108	112	—	—	—
				30	80	94	110	114	92	97	111
				50	—	—	—	—	73	85	98
	Cleveland . . .	1	2,460	30	—	—	—	—	87	97	108
				50	—	—	—	—	64	75	91
	West Penn. . .	6	3,080	20	83	92	108	109	—	—	—
				30	71	83	102	108	—	—	—
	Long Island . .	10	3,800	20	80	92	110	104	—	—	—
				30	65	80	100	99	—	—	—
	Potomac . . .	12	2,390	20	81	90	101	100	—	—	—
				30	64	75	91	93	—	—	—

TABLE 1—*continued*

(c) Durability and volume changes of fly-ash concrete

Fly ash			Fly-ash cement		No. of freezing cycles required		Length change: millionths			
							Expansion in fog	Net contraction (Storage in air after 28 days)		
Name	Carbon: %	Specific surface: sq. cm/gm	Fly ash: %	w/c ratio by weight	For increase in length of 1%	For loss in weight of 25%	28 days	35 days	56 days	6 months
			0	0·42	92	136	30	275	395	515
Commonwealth Edison	1	5,530	10	0·40	179	208	15	265	360	460
			20	0·40	284	317	20	260	355	450
Chicago	2	4,150	10	0·42	123	158	20	280	400	495
			20	0·40	208	237	25	245	380	470
Duquesne	7	2,700	10	0·44	103	146	25	275	440	520
			20	0·46	107	146	35	275	440	525
Rochester A.	8	3,850	10	0·44	88	130	25	275	440	525
			20	0·45	195	212	15	290	415	470
Dayton	9	2,980	10	0·44	202	226	10	310	450	530
			20	0·45	229	240	20	295	420	475
Detroit	12	3,220	10	0·44	176	188	20	305	455	540
			20	0·46	212	226	25	325	470	550
Philadelphia	12	2,920	10	0·43	201	210	25	295	430	535
			20	0·44	187	198	25	335	460	545
Newark	13	4,980	10	0·42	237	266	30	285	395	490
			20	0·43	344	360	35	325	435	540
Rochester B.	14	4,240	10	0·44	171	188	25	290	460	560
			20	0·45	167	187	30	285	435	520
Consolidated Edison	16	2,470	10	0·44	193	217	35	300	430	535
			20	0·46	187	200	30	340	495	585
Indiana	17	3,920	10	0·42	246	259	45	280	415	525
			20	0·43	279	307	40	295	425	515
Average for eleven fly ashes	10	3,720	10	0·43	174	200	25	285	425	520
			20	0·44	218	230	25	295	430	515

TABLE 2.—TESTS AT GLASGOW UNIVERSITY

(a) Strength tests

Ash	Mix	Fly ash: %	w/c ratio	Curing temperature	Strength (lb/sq. in.) and percentage of control					Compacting factor	Density
					7 days	28 days	90 days	6 months	12 months		
Braehead	4 : 2 : 1	0	0·60	58° to 64°F	2,850 (100)	4,780 (100)	6,800 (100)	7,850 (100)	8,800 (100)	0·89	158
		10	0·60		85	84	95	96	103	0·89	157
		20	0·60		72	73	80	92	99	0·91	157
		30	0·60		51	53	68	77	81	0·94	154
		30	0·56		62	63	79	90	96	0·87	155
		30/25	0·60		58	64	77	87	92	0·90	153
	5 : 2½ : 1	0	0·63	58° to 64°F	2,450	4,450	6,500	7,450	8,050	0·75	158
		10	0·63		94	92	95	100	101	0·76	158
		20	0·63		66	68	78	89	96	0·78	156
		30	0·63		53	54	63	75	84	0·84	157
		30/25	0·63		67	67	79	88	97	0·83	157
Portobello	4 : 2 : 1	0	0·60	58° to 64°F	2,640 (100)	4,350 (100)	6,200 (100)	7,000 (100)	7,600 (100)	0·90	155
		10	0·60		83	87	91	102	101	0·93	154
		20	0·60		59	71	86	94·5	97	0·93	153
		30	0·60		48	52	71	83·5	85	0·99	152
		30	0·56		63	69	85	98	98	0·88	155
		30/25	0·60		51	61	79	89	92	0·92	153
	5 : 2½ : 1	0	0·63	90° to 100°F	2,270 (100)	3,460 (100)	5,600 (100)	5,750 (100)	6,500 (100)	0·82	155
		10	0·63		78	95	99	103	104	0·83	156
		20	0·63		70	84	99	109	101	0·83	155
		30	0·63		59	69	86	91	95	0·84	154
		30/25	0·63		67	77	90	96	98	0·85	156

TABLE 2—(*cont.*)

(b) Setting time tests (B.S. 12)

% age cement	% age fly ash	Initial setting time: hr. min	Final setting time: hr.	Water used in setting-time test: %
100	0	3 20	7½	26·2
90	10	3 10	8	27·2
80	20	3 20	9	27·4
70	30	3 50	9½	28·0

(c) Shrinkage tests (B.S. 1881)

Proportions	Fly ash: %	w/c ratio	Total shrinkage
4 : 2 : 1	0	0·60	0·080
	10	0·60	0·0795
	20	0·60	0·080
	30	0·60	0·077
	30	0·56	0·075
	*30/25	0·60	0·070
5 : 2½ : 1	0	0·63	0·062
	10	0·63	0·075
	20	0·63	0·067
	30	0·63	0·056
	*30/25	0·63	0·069

* 30% fly ash replacing 25% cement

TABLE 3.—TYPICAL STRENGTH-TEST RESULTS

	Type of structure	Length: ft	Height: ft	Breadth: ft	Volume: cu. yd	Facing mix: lb/cu. yd					Hearting mix: lb/cu. yd					Facing		Hearting	
						Cement	Fly ash	Agg.	Air: %	Max. size agg.: in.	Cement	Fly ash	Agg.	Air: %	Max. size agg: in.	7-day: lb/sq. in.	28-day: lb/sq. in.	7-day: lb/sq. in.	28-day: lb/sq. in.
Otto Holden Dam, Ontario	Sample sections of dam	36	12	14	240	358	114	3,300	—	2½	—	—	—	—	—	2,500	4,200	—	—
		—	—	—	—	508	—	3,320	—	2½	—	—	—	—	—	3,100	3,600	—	—
Power Stations in New York and New Jersey	Foundations and circulating-water intakes	—	—	—	—	466	117	3,200	*	*	—	—	—	—	—	2,640	3,750	—	—
		—	—	—	—	583	—	3,200	*	*	—	—	—	—	—	3,000	4,020	—	—
Liberty Dam, Baltimore	Gravity dam	704	160	134·5	166,000	376	94	*	3·5	6	282	94	*	3·5	6	*	*	*	*
Palisades Dam, Idaho	Tunnel linings	27 ft-dia. and 34 ft-dia. tunnels				*					*					*	*	*	*
Hungry Horse Dam, Montana	Gravity arch-dam	2,115	564	355·5	3,000,000	283	90	3,534	3·3	6	187	90	3,652	3·9	6	2,250	4,000	1,430	2,960
Canyon Ferry Dam, Montana	Gravity dam	1,010	225	173	400,000	245	82	3,606	3·0	6	188	90	3,668	3	6	2,400	3,900	1,880	3,150
Baltimore Road	Paving slab	6-in. × 12-in. cylinders				564†	114	3,175	2·5	*	—					2,725	3,355	—	—
		and 6-in. × 9-in. cores				564	—	2,890	3·5	*	—					3,125	3,990	—	—

* Figures not available

† This mix makes more than 1 cu. yd concrete

called on to show its resistance to permeability until they are at least 1 year old. The apparatus developed in the United States for testing permeability appeared to be an improvement on that generally used in the United Kingdom and is shown in Fig. 1.

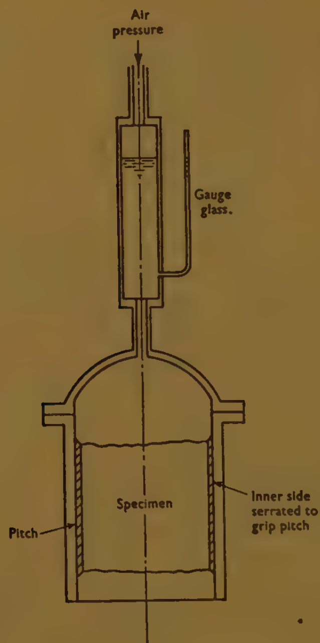

Fig. 1

Results of laboratory tests for varying mixes and different classes of fly ash are given in Table 1 and Fig. 2, and for those in Glasgow in Table 2 and Figs 3a and 3b.

Curing conditions in the United States are quite different from those in the United Kingdom, being 73°F compared with 60°F, and consequently the chemical reaction between the fly ash and lime can take place more readily. This is probably the reason why their early strength figures are higher than those in the Glasgow tests.

Examples are given below of projects in which fly ash has been used in Canada and the United States. The sites are shown in Fig. 4 and, where known, the mixes used are given in Table 3.

Examples of use of fly ash in Canada and the United States

(1) *Ontario Hydro-Electric Commission*

Field tests were carried out by the Ontario Hydro-Electric Commission on two lifts of the Otto Holden Dam. Each lift was 36 ft long × 14 ft wide × 12 ft high, a much higher lift than is usual in British practice. This experiment showed that in such lifts the temperature rise of fly-ash concrete (25% fly ash replacing cement) was

approximately 25°F less than with normal concrete, its actual value being 60°F. These temperature conditions were reproduced in their laboratory and specimens cured under them showed that after 15 days the fly-ash concrete had the same strength as a normal concrete and thereafter had greater strengths. The actual mix used in the Commission's experiments was 358 lb. of cement, 150 lb. of fly ash, 1,320 lb. of sand, 1,980 lb. of stone, and 295 lb. of water.

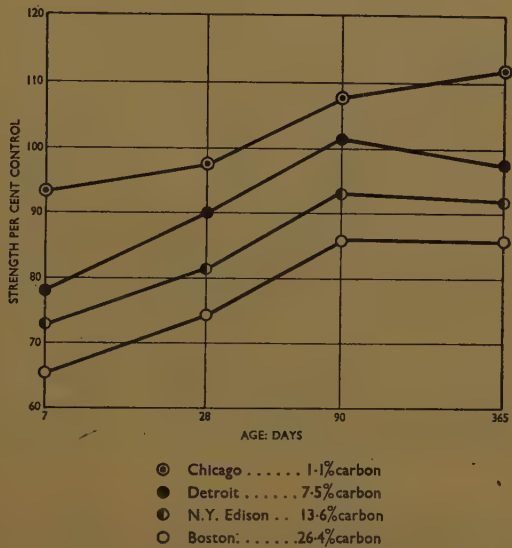

FIG. 2.—TESTS IN UNITED STATES

Notwithstanding the favourable field tests which had been carried out at the Otto Holden Dam and their aliveness to the advantages of fly ash, the Commission had not used it in the concrete for the lining of the two 45-ft tunnels for the Adam Beck No. 2 power station. The reason for not doing so was that in these large tunnels stripping time was even more important than in smaller tunnels. In this respect, fly-ash concrete was not considered to be good enough and it had been decided not to use it.

(2) *Steam power stations in New York and New Jersey*

At these power stations, fly-ash concrete had been used extensively for heavy foundation work for large steam turbo-alternators and for walls of circulating-water intakes. In some cases, these walls were at least 5 years old and had been subject to tidal variations and to salt water without showing the least sign of deterioration. Slab work and heavy reinforced concrete for the power-station structures seemed to be excellent. The concrete in all these cases had been purchased from ready-mixed-concrete firms, who provided concrete mixed to an agreed specification with approximately 16% replacement of cement by fly ash.

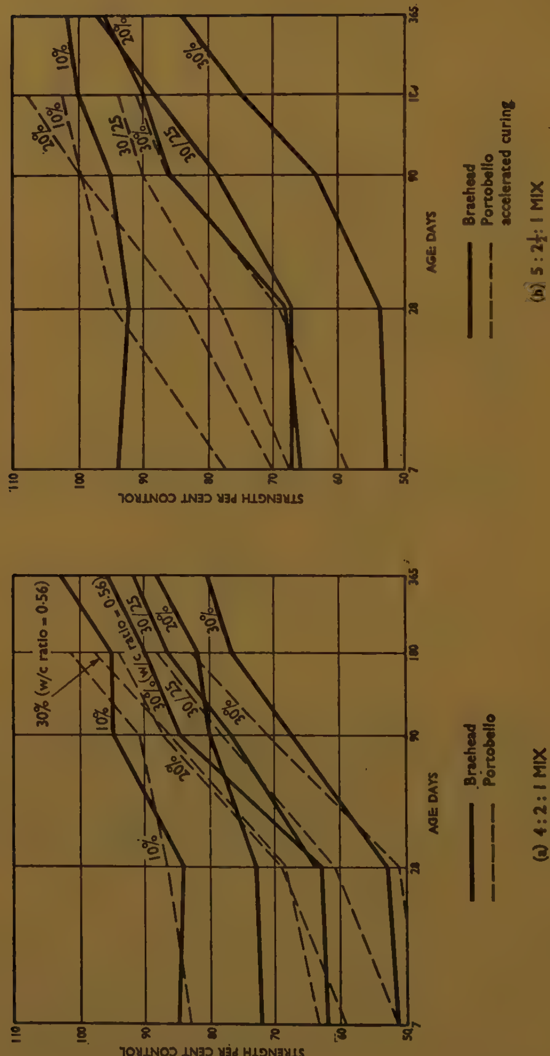

Fig. 3.—Tests in Glasgow

The fly ash used came within the requirements of the New York Building Code, which are:—

SiO_2	35% min.
Al_2O_3	15% min.
MgO	3% max.
SO_3	3% max.
Specific surface . .	3,000 sq. cm/gm.

(3) *Liberty Dam, Baltimore*

Fly-ash concrete was used throughout in the Liberty Dam at Baltimore, which was built to augment the water supply for Baltimore. It is 704 ft long, 160 ft high above ground level at its highest point, and 134·5 ft wide at the base. The face concrete had proportionately less fly ash since the mix for this contained one bag of cement more than the hearting concrete. Nearly 1 year after completion, because of delays with road diversions, the dam had not been used for impounding water. The effect of wetting could therefore not be seen. The finish and general appearance, however, were good; the only blemish seen was some crazing on the surface of the spillway crest and at the curves between the spillway steps and the dam structure. The spillway crest had been finished with a mortar which seemed to have been too rich. The intake-tower structure, which was also of fly ash, was a particularly good job. No particular difficulties had been encountered in controlling the field work although, at the outset, some alterations were needed to overcome consolidation of the fly ash in the bulk-delivery containers. Particulars provided by the City Engineer of Baltimore are given in Fig. 5 about the strength growth of fly ash in a 9-in. road paving, as determined by core boring.

(4) *Bureau of Reclamation*

Before deciding to use fly ash, the Bureau of Reclamation demonstrated by a long series of tests that careful selection and control of fly ash were essential if its use was to be justified; they selected Chicago fly ash as a sufficiently consistent material to do this. The test results obtained during construction at site not only fulfilled the Bureau's expectations but both from a strength and impermeability-with-age point of view were better.

(*a*) *Palisades Dam.*—This is a large earth-fill dam with fly-ash concrete in the tunnel linings, intake structure, spillway, and other discharge arrangements, and in the power-station foundations and structure. The inclusion of fly ash in the concrete was not popular with the site staff and every opportunity was sought to keep it out. Site tests did not disclose any failure in the routine tests of fly-ash concrete, although there were certain unaccountable discrepancies for which several reasons were advanced. The most probable was the decision to use the same weighing arrangements for cement and fly ash.

(*b*) *Hungry Horse Dam.*—This structure is 564 ft high above ground at its highest point, 2,115 ft long, and 355·5 ft wide at the base. All but the top 30 ft had been built with fly-ash concrete. No observable difference could be seen between the concretes above and below this level. Localized spalling due to inclusions of soft aggregate was a feature of this dam, but this was just as prevalent in the top 30 ft as in the rest of the dam. So far as could be ascertained, no difficulties had arisen in control of the mixes and there was no observable leakage anywhere.

(c) *Canyon Ferry Dam.*—This dam, about 1,000 ft long and 225 ft high, is of similar size to the Liberty Dam, Baltimore, and to those being built by the North of Scotland Hydro-Electric Board. Fly-ash concrete with a slightly leaner mix than at Hungry Horse Dam was used throughout. The aggregate used seemed to have been much better here, since there was no spalling as at Hungry Horse. There was, however, more leakage into the various galleries than at Hungry Horse. This had no apparent connexion, however, with the concrete used, for it seemed to come from the rock and the construction joints. Considering the nature of the tightening arrangements used in these, the amount of water seemed high. In all other respects, the work looked excellent. During construction, control of the ingredients and the concrete gave no trouble.

Unlike the Bureau of Reclamation, the Corps of Engineers had not used fly ash on any major structure. At their concrete laboratories at Jacksonville comparative studies had been made over a considerable period of various pozzolanic materials

FIG. 4

and additives such as slag, natural cements, pumicite, and calcined shales. The evidence produced in these had not convinced the Corps of Engineers that the use of fly ash as an ingredient in concrete was justified. Fly ash had not compared well with other pozzolanas for the range of tests employed, namely, strength, impermeability, frost, etc. Actually, the substitution used was 45% on a volume basis, which is unfavourable to fly ash because its specific gravity is lower than cement.

Not unnaturally, the American Portland Cement Association did not favour the substitution of fly ash for cement. Their view was that there were too many uncertainties and it was possible to get fly ash which would not behave as desired.

Their chemist was of opinion that fly ash did not improve the resistance to aggressive waters, his argument being that even though free lime did combine with silica the calcium silicate so formed tended to be nearly as soluble as free lime.

When fly ash is used in concrete it is essential that, as received, it should come within the limits set as a result of laboratory experiments. The Bureau of Reclamation restricted their source of supply to Chicago and to reach the dam sites at Palisades (Idaho), Hungry Horse, and Canyon Ferry (Montana), long haulage by the railroad was involved. Delivery was by bulk with air agitation to prevent consolidation. Site tests on arrival were, however, largely dispensed with since the fly ash was tested before dispatch. No form of colour-scale tests was in use although these are considered as a useful guide to the carbon content of a fly ash. Fly ash tends to be unpopular with site staffs since it adds to the already numerous control problems at the site. It had been found that any variation in its carbon content has an immediate effect on the water/cement ratio and the volume of air entrainment, thus making control more complicated for the concrete inspector.

Site testing of concrete has always suffered from the disadvantage that it is not possible to forecast with certainty the growth of strength with time. This becomes an even more serious problem with concretes where a proportion of the cement is replaced by a slow-strengthening material such as fly ash. It is recognized that such concretes start off with a low initial strength, but it is essential that the long-term strength should be satisfactory. Any means of anticipating what is likely to happen in this respect is therefore of great value. At the moment it looks as if some form of accelerated curing might provide the answer. Tests have been made at Glasgow University using a 24-hour curing period in hot water and the most promising give the results shown in Table 4.

TABLE 4

Mix	Fly ash: %	Strength at 24 hours: lb/sq. in.	7-day strength: lb/sq. in.
4 : 2 : 1 (w/c = 0·60)	0	1,830 (79·5%)*	2,300
	20	1,170 (76%)	1,410
5 : 2½ : 1 (w/c = 0·63)	0	1,520 (78%)	1,950
	20	1,000 (77%)	1,300

* The percentage figures given are the ratio of the accelerated strength to the 7-day strength.

Note.—The cubes were kept at room temperature for 2 hours and then placed in water at 140°F for 22 hours.

EXPERIENCE IN THE UNITED KINGDOM

Two contracts for dam structures are now in progress for which the North of Scotland Hydro-Electric Board have decided to use fly ash on a 20% replacement-

of-cement basis. The first is at the Lednock dam on the St Fillans section of the Breadalbane scheme, where already 16,000 cu. yd of this kind of concrete have been placed. The second is at the Lubreoch dam on the Killin section of the Breadalbane scheme, where necessary additional equipment is being added to the mixing plant to enable fly ash to be added.

OTHER MATERIALS

Similar researches to that for fly ash have been done for other substitutions such as slag, natural cements, pumicite, and calcined shales. They too have had for their object a reduction in cement content without any loss in the ultimate strength of the concrete, by depending on the long-term reaction between silica and free lime. Professor Davis of the University of California, who is one of the original and principal investigators in the United States of the use of pozzolanas, is of opinion that few of the other pozzolanas are likely to be able to compete with fly ash, except where they have other special advantages, because they cannot be used as found without further treatment. This may take the form of calcining or grinding, the cost of which would make them appreciably more expensive than fly ash.

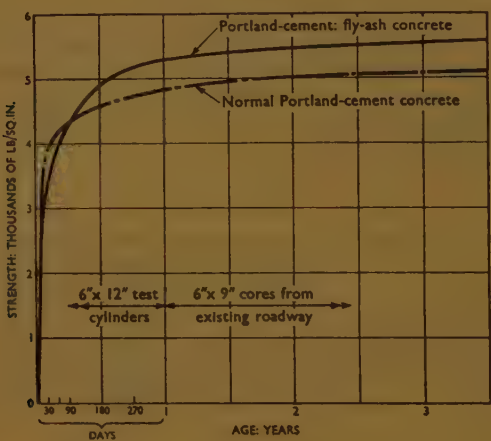

FIG. 5.—CURVE SHOWING RELATIVE STRENGTH OF PORTLAND-CEMENT: FLY-ASH CONCRETE VERSUS NORMAL PORTLAND-CEMENT CONCRETE FOR 9-IN. ROADWAY PAVEMENT PLACED IN OCTOBER 1950 AT COOK'S LANE, BALTIMORE, U.S.A.

The North of Scotland Hydro-Electric Board have investigated two such substitutions, namely, blast-furnace slag and diatomite. The former is fundamentally the Trief process in which a considerable percentage of the cement is replaced by suitable blast-furnace slag which has been finely ground by the wet process. Tests at the Royal Technical College showed that sufficient strength could be obtained for mortar in which 50% of the cement had been replaced by finely ground slag from Colville's

It was decided to use this process of cement replacement in the Cluanie Dam of the Moriston scheme of the North of Scotland Hydro-Electric Board. Since the start of the contract in 1952, approximately 180,000 cu. yd of 70%-cement-replacement

concrete have been used. The slag for Moriston is obtained from Colville's works near Glasgow and is wet ground on the site.

It has also been used at the Loyne Dam and for lining the 12-ft-dia. and 14,000-ft-long power tunnel at Cluanie. Other tunnels in the Breadalbane scheme are to be lined with it. In all, about 225,000 cu. yd of concrete have so far been placed using this process.

The average cube strength obtained at Moriston for 70%-cement-replacement 7 : 1 hearting concrete at 28 days was 2,940 lb/sq. in.; the strength specified before it was decided to adopt Trief for the same mix, but using 100% Portland cement, was 2,250 lb/sq. in.

On the Continent, the Trief process has been used in dam construction at Bort-les-Orgues by Électricité de France. This dam is 392 ft high × 1,280 ft long and contains 863,000 cu. yd of concrete.

The use of Scottish diatomite as a cement replacement has been investigated at Glasgow University. The price of this material, when processed, is approximately five times that of cement; consequently, unless other outstanding virtues can be found, it can only be an economic substitution if it replaces about five times its weight in cement. Tests show that if 20% of the cement is replaced by 4% diatomite, the strength of the latter concrete after 6 months is 85% of a control mix of the same workability. Professor Davis has found, however, that diatomite produces better comparative figures when used in conjunction with air entrainment.

Air entrainment

In considering the use of fly ash, one feature which has made it difficult to compare American experience with that in the United Kingdom has been the almost universal use in the United States of air entrainment in concrete. Its introduction was first of all brought about as a means of increasing frost resistance, but it is now used almost everywhere to improve workability as well. As already mentioned, it has been found that small variations in carbon content of fly ash affect materially the quantity of air-entraining agent needed for a given percentage of air entrainment. This is why fly ash needs to be so consistent in quality in the United States, with a low percentage of unburnt carbon. The use of air entrainment, which gives an improvement in workability, tends to hide any merits which fly ash may have in this respect.

Conclusion

The general conclusion that can be drawn is that if the concrete is not required to withstand its working load until some time after pouring—and in most mass structures this is the case—then 20% of the cement can be replaced by fly ash. The resulting concrete will be more workable, have equal or better resistance to weathering, and after 6 to 12 months will be as strong as the normal concrete.

It is generally specified that the fly ash must be of low carbon content and high specific surface. These requirements are advantageous but a recent Paper by Minnick,[2] the experience in the Eastern States of America, and at Lednock, all seem to show that they are not absolutely essential.

There must, however, be a yardstick by which the quality of the site concrete can be judged and this standard must be drawn up from laboratory experiments. It is,

[2] L. J. Minnick, "Investigations relating to the use of fly-ash as a pozzolanic material and as an admixture in Portland cement concrete." A.S.T.M., vol. 54, p. 1129.

therefore, highly desirable that the same type of fly ash should be used both for the laboratory and the site. This means consistency in the source of supply, which is very unlikely since the coal burned at power stations is likely to be obtained from many sources.

The standard, therefore, should be such that judgement can be quickly made. The usual 28-day-strength requirements are poor in this respect and attention is now being given to the development of accelerated tests.

The Paper, which was received on 4 June, 1956, is accompanied by five sheets of diagrams from which the Figures in the text have been prepared.

Correspondence on this Paper should be forwarded to reach the Institution before 15 February, 1957. Contributions should not exceed 1,200 words.—Sec.

ELECTION OF ASSOCIATE MEMBERS

The Council at their meeting on 18 September, 1956, in accordance with By-law 14, declared that the following had been duly elected as Associate Members:

ALDERSON, KENNETH JORDAN, B.Sc. (*Durham*), Grad.I.C.E.
ALLISON, HUGH BRUCE, Grad.I.C.E.
ANDERSON, NOEL BARBER, B.Sc. (*Edinburgh*), Grad.I.C.E.
ARMSTRONG, MALCOLM DOUGLAS, B.Sc. (Eng.) (*London*).
ASHTON, JOHN ANTHONY, B.Sc. (*Leeds*), Grad.I.C.E.
BEDFORD, LANCE DYNES, Stud.I.C.E.
BEVIS, WILLIAM JAMES, B.E. (*New Zealand*), Grad.I.C.E.
BRAIDE, THOMAS SAMUEL.
BRISTOW, WILLIAM KENNETH, Grad.I.C.E.
BROWN, ROBERT HENRY, B.Sc.(Eng.) (*London*), Grad.I.C.E.
BRUNSKILL, DENNIS MICHAEL, Grad.I.C.E.
BUXTON, FREDERICK NOEL, B.E. (*New Zealand*).
CHAMPION, WILLIAM DAVID, B.E. (*New Zealand*), Grad.I.C.E.
COCHRANE, HUGH BENJAMIN, M.A. (*Cantab.*), Grad.I.C.E.
COTTON, HENRY MALBY, B.E. (*New Zealand*).
CRAIG, CAMERON, Grad.I.C.E.
CRAIG, DOUGLAS MALCOLM, B.Sc. (*St Andrews*), Grad.I.C.E.
CROWTHER, LESLIE, B.Sc.Tech. (*Manchester*).
DAVIS, JOHN ALBERT, B.Sc.(Eng.) (*London*), Grad.I.C.E.
DEED, EDWARD HENRY, B.Sc.(Eng.) (*London*), Grad.I.C.E.
EDMONDS, NORMAN, Stud.I.C.E.
EDWARDS, ARTHUR DAVID, B.Sc.(Eng.) (*London*), Grad.I.C.E.
FARQUHARSON, IAN STUART REID, B.A., B.A.I. (*Dublin*), Grad.I.C.E.
FISHER, JOHN HUGH BENNETT, B.E. (*New Zealand*), Grad.I.C.E.
FRANKLIN, DEREK WILLIAM, M.A. (*Cantab.*), Grad.I.C.E.
GAHAN, DERMOT FRANCIS, B.Sc.Tech. (*Manchester*), Grad.I.C.E.
GOLDSBROUGH, DEREK HOWARD, B.Sc. (Eng.) (*London*), Grad.I.C.E.
GOLDSBRO', PETER GEORGE, Grad.I.C.E.
GREGSON, JOHN KEITH, B.Sc. (*Nottingham*), Grad.I.C.E.
GRUMMITT, CLAUDE NORMAN, Grad.I.C.E.
HARVEY, PERCY, Grad.I.C.E.
HILL, MARTIN ROBERT EWBANK, Grad. I.C.E.
HOGG, HENRY MOLLISON, B.Sc. (*Glasgow*), Grad.I.C.E.
HUTCHINSON, JULIUS KENNETH BEVILLE, Grad.I.C.E.
JEFFS, GEOFFREY AYLING, Grad.I.C.E.
JONES, DANIEL EDWARD REECE, B.Sc. (Eng.) (*London*), Grad.I.C.E.
JONES, ROY, B.Eng. (*Sheffield*).
KENNEDY, GORDON ALEXANDER, B.Eng. (*Liverpool*).
LATHAN, JOSEPH RICHARD, B.Sc.(Eng.) (*London*), Stud.I.C.E.
LLEWELYN, FREDERICK JOHN, B.Sc. (*Wales*), Grad.I.C.E.
MACINNES, ANDREW JAMIESON, Grad. I.C.E.
MCCREATH, GEORGE FORBES, B.Sc. (*Glasgow*), Grad.I.C.E.
MCKENDRY, DANIEL JOSEPH CALVIL, B.Sc. (*Belfast*), Stud.I.C.E.
MCKIBBIN, THEODORE GEORGE HENRY, B.Sc. (*Belfast*), Grad.I.C.E.
MCKILLOP, COLIN, B.Sc. (*Glasgow*), Grad. I.C.E.
MCLEAY, RONALD MURDOCH, B.E. (*New Zealand*).
MACRAE, KENNETH, B.Sc. (*Glasgow*).
MARTIN, LAURENCE HAROLD, B.Sc. (*Leeds*), Grad.I.C.E.
MEADE-KING, RICHARD OLIVER, B.A. (*Cantab.*), Grad.I.C.E.
METELERKAMP, DAVID RAWSTORNE, Grad. I.C.E.
MONTGOMERY, REX WALTON, B.E. (*New Zealand*), Grad.I.C.E.
MORISON, RONALD STEWART, B.E. (*New Zealand*), Stud.I.C.E.
MORRIS, PHILIP HENRY, B.Sc.(Eng.) (*London*), Grad.I.C.E.
MORRIS, WILLIAM JAMES, B.Sc.Tech. (*Manchester*), Grad.I.C.E.
MUGGLESTONE, JAMES CYRIL, B.Sc. (Eng.) (*London*), Grad.I.C.E.
MURPHY, JAMES SMITH, Grad.I.C.E.
OLIPHANT, DAVID NICHOLSON, B.Sc. (*Edinburgh*), Grad.I.C.E.
OLIPHANT, EBENEZER, Grad.I.C.E.

O'Hanlon, Henry McCann, B.Sc. (*Glasgow*), Grad.I.C.E.
Olliver, Peter Russell, B.Sc.(Eng.) (*London*), Grad.I.C.E.
Palmer, Michael Alan, B.Eng. (*Sheffield*), Grad.I.C.E.
Phillips, Alan, B.Sc.(Eng.) (*London*), Grad.I.C.E.
Pilling, James Hugh, B.Sc.Tech. (*Manchester*), Grad.I.C.E.
Poole, Clifford Jerrold, Grad.I.C.E.
Powell, Arthur Robert, Grad.I.C.E.
Ray, William Joseph Fergus, B.A. (*Oxon*), Grad.I.C.E.
Reily, Denis, B.A. (*Cantab.*), Grad.I.C.E.
Renner, Karl Simmons, B.E. (*New Zealand*), Stud.I.C.E.
Ring, Basil Albert, Grad.I.C.E.
Robinson, Barry Norman, B.Sc. (*Glasgow*), Grad.I.C.E.
Robinson, Norman.
Roe, John, B.E. (*Queensland*), Grad.I.C.E.
Scarlett, John Hall, B.Sc.(Eng.) (*London*), Grad.I.C.E.
Shapland, Christopher Robert, Grad. I.C.E.
Sinclair, Brian Mowlem, Grad.I.C.E.
Smith, Albert Charles, B.Sc.(Eng.) (*London*), Grad.I.C.E.
Tait, Graham Anderson.
Thomas, Austin Withiel, B.E. (*New Zealand*), Grad.I.C.E.
Turner, Douglas Arnold, B.Sc.(Eng.) (*London*).
Turner, Geoffrey, Grad.I.C.E.
Turner, Leonard Andrew Bruce, Grad. I.C.E.
Walkey, John Meager, B.Sc.(Eng.) (*London*), Grad.I.C.E.
Wex, Bernard Patrick, B.Sc.(Eng.) (*London*).
Whidborne, Richard Claude.
Whittaker, John Haslop, B.Sc.(Eng.) (*London*), Grad.I.C.E.
Williams, Idwal, Grad.I.C.E.

DEATHS

It is with deep regret that intimation of the death of the following has been received.

Members

Leonard Andrews, M.B.E. (E. 1903, T. 1910).
Arthur Langtry Bell, M.Sc., B.A. (E. 1904, T. 1913).
Harold Edward Midgley, M.A. (E. 1905, T. 1919).
Langdon Pearse, M.Sc. (E. 1924).

Associate Members

Francis Cameron Bridge, B.Sc.(Eng.) (E. 1932).
Professor Charles Henry Bulleid, O.B.E., M.A. (E. 1910).
William Arthur Burton, O.B.E. (E. 1908).
Edward Huntingdon Leaf, B.A. (E. 1898).
Harry Leopold Lloyd (E. 1925).
Colonel Archibald George McDonald, O.B.E., T.D., B.Sc.(Eng.) (E. 1927).
Henry Furney Nolans, M.A., B.A.I. (E. 1907).
Thomas Clement Ormiston-Chant (E. 1998).
Richard Basil Sargent (E. 1935).
James Zeno Sloan, B.E. (E. 1917).
George Bruce Tomes (E. 1896).
Sidney George Winn (E. 1911).
Richard Wormell (E. 1919).

OBITUARY

RICHARD JOSEPH HOWLEY, C.B.E., B.E., who was born on 9 July, 1871, died in April 1955.

He was educated at Oscott College and the Royal University of Ireland (now known as the National University of Ireland). In 1942 the University awarded him the degree of B.E. (Non-causa).

After serving as a pupil engineer on the staff of Messrs Townsend and G. H. Ryan from 1891 until 1895, Mr Howley was engaged for several years on railway and dock construction work. In 1899 he joined the British Electric Traction Co., as Permanent Way Engineer. He was Joint Manager of the Company from 1912 to 1919, and Chairman from 1942 to 1946. He was President of the British Electrical Federation Ltd.

Mr Howley specialized in the establishment and operation of passenger transport system providing railway, tramway, and omnibus services. He acted on many occasions as an Arbitrator and Expert Witness.

In 1919 he was appointed a Commander of the Order of the British Empire for his work as a member of the Tramways (Board of Trade) Committee and the Railways Priority Committee during the 1914–18 war.

In 1897 he presented a Paper[1] to the Institution on "Railway construction through bog-land".

Mr Howley was elected an Associate Member in 1897, and was transferred to the class of Members in 1912. He was also a Member of the Institute of Transport.

SIR JOHN ROBERT KEMP, M.E., who was born on the 6th October, 1883, died on 28 February, 1956.

He was educated at Victorian State Schools, Melbourne Technical College, and Melbourne University.

After an engineering cadetship in the Public Works Department, Victoria, he joined the Patents Office, where he became an Assistant Examiner. From 1910–13 he was a Municipal and Consulting Engineer. The Victorian County Roads Board was formed in 1913 and he served as a senior engineer from 1913–20. After serving as Chairman of the Main Roads Board, Queensland, from 1920–25 he was subsequently appointed Commissioner for Main Roads, a position which he held from 1925–49.

He was appointed a member of the Bureau of Industry, Queensland, and Chairman of its Construction Boards for Bridges, Reservoirs, and the building of the University. Sir John was Chairman of the Royal Commission on Electricity, Queensland, 1936–37, and of the Royal Commission on Transport in the same period.

In 1937 he was elected a Fellow of the Australian and New Zealand Association for the Advancement of Science, and in 1953 an Honorary Fellow of The Australian Institute of Management. He also served a term as President of the Institution of Engineers, Australia.

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[1] Min. Proc. Instn Civ. Engrs, vol. 132 (1897–98, Pt II), p. 322.

He was Australian Government representative at the International Road Congress, The Hague, in 1938. From 1939 to 1945 Sir John was Deputy-Director-General Allied Works Council, and from 1939 to 1954 was Co-ordinator General of Public Works for Queensland.

He was elected a Senator of Queensland University in 1944, and from that date until 1954 he was also Chairman of the Bureau of Investigation of Land and Water Reserves of Queensland.

The National Security Resources Board was formed in 1950 by the Prime Minister of Australia and Sir John was one of its first members.

In 1932 he was awarded the Kernot Memorial Medal of Melbourne University, and in 1942 the Peter Nicol Russell Medal for distinguished engineering service.

He was elected Member in 1929.

He is survived by his widow, Lady Ann Kemp.

SPECIAL GENERAL MEETING

Thursday, 6th September, 1956

WILLIAM KELLY WALLACE, C.B.E., President, in the Chair

The Notice convening the meeting was taken as read.

The President drew the attention of the meeting to the fact that under By-law 109 no resolution other than the resolutions as circulated for the purpose of the postal vote and no amendment or variation of any such resolution shall be proposed or voted upon at the meeting.

The President then announced that any member present who had not already recorded his vote by post could vote in person at the meeting, and that any member who might wish to do so could withdraw his recorded vote and vote in person at the meeting provided that he withdrew the whole of his voting paper and voted personally upon each resolution as it was dealt with.

No member indicated a wish to withdraw his postal vote.

The President then moved Resolution No. 1:

RESOLUTION No. 1.

"That, the undermentioned rates of Subscription, entrance fees and life composition fee, having been fixed by the Council, be and are hereby confirmed, and that the same do come into force on the 1st January, 1957.

	Class A (London area) £	Class B: Elsewhere in the British Isles and Abroad £
Members	11	8
Associate Members	7	6
Members and Associate Members (retired)	3	3
Associates	7	7
Graduates	5	5
Students	2	2
Entrance Fee on election of a Member		£60
Entrance Fee on election of an Associate Member who has not been a Graduate or Student of the Institution		£15
Entrance Fee on election of an Associate Member who has been a Graduate or Student of the Institution		£10
Entrance Fee on election of an Associate		£15
Transfer Fee on transfer of an Associate Member to Membership		Nil
Life Composition Fee		£200"

He announced that the postal votes received totalled 3,451 in favour of the resolution, and 1,104 against.

The resolution was then put to the meeting and no additional votes for or against were recorded. The President then announced that the final voting on Resolution No. 1 was

For:	3,451
Against:	1,104

and declared that the resolution was carried.

The President then moved Resolution No. 2:

RESOLUTION No. 2.

"'That the By-law 84, be altered by substituting the date '31st December' for the date '31st March'."

and announced that the postal votes totalled 4,420 for the resolution and 135 against.

Mr T. N. C. Bulman asked if, in the event of Resolution No. 2 being carried, the Annual General Meeting could be advanced from its usual date in June to an earlier date in order to encourage a better attendance at that meeting.

The President replied that the question did not arise out of the resolution, but that it would be considered by the Council.

No additional votes for or against were recorded by the members present, and the President announced that the total vote on the resolution was

For:	4,420
Against:	135

and declared that the resolution was carried, adding that presumably the 10,000 members who recorded no vote were content to leave the matter in the hands of the Council.

The meeting then terminated.